

ACI 533R-11

Guide for Precast Concrete Wall Panels

Reported by ACI Committee 533



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Guide for Precast Concrete Wall Panels

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American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
U.S.A.
Phone: 248-848-3700
Fax: 248-848-3701

www.concrete.org

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Guide for Precast Concrete Wall Panels

Reported by ACI Committee 533

Brian D. Miller, Chair

David Wan, Secretary

George F. Baty
Harry A. Chambers
Aaron W. Fink
Sidney Freedman
Harry A. Gleich
Allan R. Kenney
Benjamin Lavon

Donald F. Meinheit
Larbi M. Sennour
Venkatesh Seshappa
Ava Shypula

Thomas J. Grisinger, Consulting Member
Weilan Song, Consulting Member

This guide presents recommendations for precast concrete wall panels. It should be used with ACI 318-08, "Building Code Requirements for Structural Reinforced Concrete," which is legally binding when adopted by the local authority. This guide discusses the basic principles of design, tolerances, materials, fabrication, installation, quality requirements, and testing.

Keywords: admixtures; aggregates; architectural concrete; coatings; cracking (fracturing); curing; deflection; design; drying shrinkage; fabrication; formwork; inspection; installation joints (junction); precast concrete panels; repairs; sandwich panels; sealants; structural design; surface defects; tolerances; volume change.

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Reference to this document shall not be made in contract documents. If items found in this document are desired by the Architect/Engineer to be a part of the contract documents, they shall be restated in mandatory language for incorporation by the Architect/Engineer.

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CHAPTER 1—GENERAL CONSIDERATIONS

1.1—Introduction

The widespread popularity of concrete as a building material can be attributed to the availability, favorable properties, and geographic distribution of its naturally-occurring mineral constituents. Concrete is easily formed and molded, comparatively economical, and durable in its finished state. Architectural precast panel use has increased because of the nature of concrete as a material and the fact that prefabricated components add to construction efficiency. By exposing decorative aggregates, using veneer facing materials, and by varying sizes, shapes, and textures of panels, the engineer/architect can respond creatively to client needs.

1.2—Scope

This document provides guidelines for specifying, planning, designing, fabrication, and erecting precast concrete wall panels. Although the focus is on precast wall panels produced in established precasting plants, site precasting is an option that has been used successfully. Tilt-up concrete, as discussed in ACI 551.1R-05, is a variation of site precasting. ACI 533R should aid in establishing and maintaining quality

site fabrication as well as plant fabrication of precast wall panels.

This guide covers non-load-bearing or load-bearing panels, fabricated of normal or lightweight concrete. Panels may be one of the following types:

- Solid
- Insulated (sandwich)
- Ribbed
- Hollow-core
- Sculptured

In addition to reinforced panels, lightly prestressed (effective prestress, after all losses, between 150 and 225 psi [1.0 and 1.7 MPa]) and prestressed panels are covered. Structural design considerations addressed in Chapter 3 include the use of panels as shear wall components.

Emphasis is placed on wall panels with an integral exposed aggregate concrete surface finish. Smooth wall panels and panels with a textured or shaped architectural surface finish are included. Panels having natural stone veneer or ceramic veneer finishes are not covered in detail.

1.3—Responsibility

1.3.1 General—Contractual agreements should assign responsibilities to avoid disagreements on basic definitions and decisions originating from the specifying agency. A special report of an ad hoc committee for the responsibility for design of precast concrete structures was published by the Precast/Prestressed Concrete Institute (PCI 1988) and recommends assignment of authority and responsibility for design and construction of precast concrete structures.

This guide covers the design of panels by an engineer/architect. ACI Committee 533 presents supplemental design guidelines that are special to precast concrete wall panels and should be used with ACI 318-08. ACI 318-08 provides minimum design requirements and is legally binding when adopted by the local authority.

Overlapping responsibilities for the structural design of wall panels may introduce conflicts between the engineer/architect and general contractor regarding contract document review, design for handling, installation stresses, in-place loads, and adequacy of connections. It is essential that work assignments and responsibilities be clearly defined in the contractual arrangements.

1.3.2 Structural design—The engineer/architect can benefit from preconstruction contact with panel producers. Handling and installation procedures vary widely, and guidelines for these operations should correspond with local practices but be consistent with Chapter 3 of this guide. Most precasters maintain an engineering staff to prepare contract documents and the engineer/architect should interact with this group to obtain constructive advice and suggestions concerning local practice, fabrication details, and fabrication capabilities. When possible, this discussion should take place during the initial design phases of a construction project. Once a job is released for bidding and the structural concepts have been established, changes may be difficult to implement.

1.3.3 Reinforcement for handling and installation—Generally, the engineer/architect relies on the manufacturer for developing handling techniques and providing any additional reinforcement required for withstanding handling or installation stresses. The engineer/architect may wish to review calculations for handling stresses.

Contract documents may require the manufacturer to accept responsibility for design of panels to resist the loads shown and to resist other loads that occur during stripping, handling, shipping, and installation. In this case, it is common for contract documents to require that design calculations and installation drawings provided by the panel manufacturer be signed by a professional engineer who is either retained or employed by the manufacturer. When the load information in the contract documents is insufficient, the manufacturer should ask for additional information from the engineer of record.

1.3.4 Adequacy of connections—Contract drawings prepared by the engineer/architect should show the connections required and the load support points in sufficient detail to permit construction. During the preparation of shop drawings, manufacturers should be given the opportunity to redesign the connections if redesign will achieve economical details that facilitate manufacture or installation. If connections are designed by the engineer/architect, the manufacturer should review the connections for structural adequacy and economy. If connections are redesigned or other problems are noted, these should be brought to the attention of the engineer/architect for review. Any deviation from or discrepancy in the approved installation drawings should be communicated to the general contractor by the installation contractor before installation. The general contractor should make all necessary arrangements for corrections to be made by the parties involved before installation.

1.3.5 Handling and installation responsibilities—Responsibility for panel installation and cleaning, joint treatment, and supply of hardware for handling, attachment, and bracing should be clearly defined in contract documents. The specifier should not prescribe specific subcontractors in the document specifications. General contractors are generally more knowledgeable of the skills and experience of subcontractors who can perform the services and can more easily evaluate the alternatives.

1.3.5.1 Cleaning—Specifications should require clean panels after installation and cleaning need not be the object of a separate operation. The precast manufacturer, carrier, or both, are responsible for delivering clean panels (refer to Section 7.6). After panel installation, protecting panels from soiling and staining during subsequent operations should be the responsibility of the general contractor.

1.3.5.2 Furnishing attachment and handling hardware—Clip angles, inserts, bolts, and miscellaneous metal items are required for construction with precast panels. These items may be attached to the building frame, embedded in the precast panel for erector or for other trades, or provided loose at the job site for connection purposes.

Responsibility for supplying items to be attached to or placed in the structure to receive precast concrete units

depends on structure type and local practice. Contractual agreements should indicate who is responsible for the supply and installation of hardware. When the supporting frame is structural steel, installation hardware is normally supplied and installed by the precast erector or steel fabricator. When the building frame consists of cast-in-place concrete, hardware is normally supplied by the precast manufacturer and placed by the concrete contractor. Detailed hardware layout is prepared by the precast manufacturer for approval by the engineer/architect. Occasionally certain special inserts or sleeves are required for other trades. In these instances, the trade involved is responsible for having such parts approved and delivered to the panel manufacturer in time for embedment in the wall panels. These should be accompanied by the engineer/architect's approved placement drawings and instructions for installation.

1.3.5.3 Execution of connections—The general contractor is responsible for accurately constructing bearing surfaces and anchorages for precast elements. When a panel cannot be erected within tolerances specified in the contract documents, the matter should be called to the engineer/architect's attention for consideration and correction.

Changes, other than adjustments within the prescribed tolerances, can only be made after approval. Any adjustments affecting structural performance should be approved by the engineer of record. No panel should be left in an unsafe support condition.

1.3.6 Shop drawing approval—Installation and shape drawings prepared by the precast manufacturer (refer to Section 6.1) should be forwarded to the general contractor for approval regarding constructibility and then forwarded to the engineer/architect who checks for conformance with the design requirements and contract documents. Drawings reviewed by the engineer/architect should be returned to the manufacturer with a statement similar to one of the following:

- Reviewed for conformance with the contract documents. No resubmissions necessary.
- Reviewed, as noted, for conformance with the contract documents. No resubmissions necessary.
- Revise and resubmit.
- Rejected.

1.4—Aesthetic considerations

Fabrication techniques and procedures covered in this guide allow flexibility during fabrication to achieve uniform aesthetic results and concrete quality. Performance specifications for the appearance of precast wall panels explaining aesthetic requirements or establishing understandable criteria for acceptance are difficult to achieve. It is recommended that reference samples be used in determining product characteristics and quality, rather than writing restrictions that are difficult to achieve and may prohibit the manufacturer from using a process that offers the best possibility of producing the desired panel.

1.4.1 Design reference samples—Full-size sample panels are preferred, but construction specifications may require that the color and texture match small samples. Such samples

should be at least 12 x 12 in. (305 x 305 mm), although larger samples may be desirable. If both panel faces are to be exposed, samples should show the finished interior surface as well as the exterior face of the precast.

The manufacturer should submit samples to the general contractor for approval by the engineer/architect, while retaining duplicate samples. If the sample is not approved, resubmissions should be made until approval is obtained. Sample approval should be in writing with reference to the correct sample code number, or the approval may be written on the sample itself.

1.4.2 Range samples—At least three full-scale (but not necessarily full-size) sample panels should be specified. These sample panels should contain typical cast-in inserts, reinforcing steel, and plates as required for the project. These panels should establish the range of acceptability with respect to color and texture variations, surface defects, and overall appearance. It should be clearly stated in the contract documents how long the full-scale samples should be kept at the point of manufacture (precasting plant) or at the job site for comparison. Approved full-scale panels should be allowed to be used in the completed structure. If full-scale samples are required before, or at the beginning of, fabrication, lead time is necessary and the construction schedule should be adjusted accordingly. When full-scale sample panels are not specified, the first fabrication panels should be submitted for inspection and approval by the engineer/architect.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

b	=	width of cross section
f'_c	=	concrete compressive strength specified at age considered during design
h_{eff}	=	effective thickness of member
I_g	=	moment of inertia of gross concrete section neglecting reinforcement
k	=	effective length factor
ℓ	=	length of span
r	=	radius of gyration of cross section
ℓ_u	=	unsupported length of panel

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, “ACI Concrete Terminology,” (<http://terminology.concrete.org>). Definitions provided herein complement that resource.

bowing—the deviation of the edge or surface of a planer wall member, in the out-of-plane direction, from a line passing through any two corners of the member

camber—a deflection that is intentionally built into a structural element or form to improve appearance or to nullify the deflection of the element under the effects of loads, shrinkage, and creep.

surface out-of-planeness (roughness)—a local smoothness variation rather than a bowing variation. The tolerance for this variation is usually expressed in fractions of an inch (millimeter) or in inches per 10 ft (millimeters per 3 m).

Although other criteria are available, the tolerance is typically checked with a 10 ft (3 m) straightedge or equivalent and definition of roughness.

tolerance—a permitted variation from the basic dimension or quantity as in the length, width, or depth of a member.

unequal bowing—may be observable when panels are viewed together on the completed structure. When two panels bow in the same direction, the magnitude of unequal bowing is determined by subtracting one bowing value from another. When panels bow in opposite directions, the convex bowing is taken as positive (+) and concave bowing is taken as negative (−) by a standard sign convention, the unequal bowing is the algebraic difference.

warping—deviation of a planer surface due to displacement of one corner in relation to any two adjacent corners. Warping tolerances are stated in terms of the magnitude of the corner variation. This value is usually given in terms of the allowable variation per 1 ft (300 mm) of distance from the nearest adjacent corner with a not-to-exceed maximum value of corner warping.

CHAPTER 3—WALL PANEL DESIGN

3.1—Introduction

This guide presents design recommendations for prestressed and conventionally reinforced concrete wall panels. Load-bearing and non-load-bearing panels are covered. Precast wall panels can be differentiated on the basis of structural function as well as panel configuration. The classes and types of panels covered in this guide are defined in the following.

3.1.1 Panel classes

3.1.1.1 Non-load-bearing panel (cladding)—A precast wall panel that transfers negligible load from other elements of the structure. This panel is generally designed as a closure panel and should resist all applicable service and factored loads from wind forces, earthquake-induced forces, thermally-induced forces, forces from time-dependent deformations, self-weight, and those forces resulting from handling, storage, transportation, and installation.

3.1.1.2 Load-bearing panel—A precast wall panel designed to carry loads from one structural element to other structural elements. Load-bearing panels should interact with other panels and the supporting structural frame to resist all applicable design loads in addition to those listed for non-load-bearing panels. Load-bearing panels also include panels designed to function as shear walls.

3.1.2 Panel types

3.1.2.1 Solid panel—A panel of constant thickness; an allowance for surface texture should be made in determining effective thickness.

3.1.2.2 Hollow-core panel—Precast panel that has voids within the thickness in one direction for the full length of the panel.

3.1.2.3 Sandwich panel—Precast panel consisting of two layers of concrete separated by a nonstructural insulating core.

3.1.2.4 Ribbed panel—Precast panel consisting of a concrete slab reinforced by a system of ribs in one or two directions.

3.2—Design guidelines

3.2.1 General—Precast wall panels should be designed according to ACI 318-08 except as modified in Sections 3.2.3, 3.2.4.2, 3.2.5, 3.3, 3.4.2, 3.5.2, and 3.5.3 of this guide. ACI 318-08 requirements may be legally binding.

3.2.2 Forces for design—Where applicable, precast wall panels should be designed to resist the following forces due to:

- Construction, handling, storage, and transportation.
- Installation and impact.
- Gravity dead and live loads and lateral loads from soil.
- Hydrostatic pressure, wind, and seismic action.

Precast panels should also be designed to resist the forces developed by the following:

- Differential support settlement.
- Deformations from creep and shrinkage.
- Structural restraint.
- Effects of environmental temperatures.
- Thermal movement or bowing.
- Volume change of the panel with respect to the supporting structure.
- Local stress concentrations in the vicinity of connections and applied loads.

3.2.3 ACI 318-08 provisions applicable for member design—The following chapters or sections of ACI 318-08 should be followed for the design aspects enumerated, except as otherwise modified in this guide.

Effective prestress—Section 18.6. The average concrete stress due to prestressing after losses is limited to a range of 150 to 800 psi (1.0 to 5.5 MPa).

Flexure—Chapter 10 for nonprestressed panels and Chapter 18 for prestressed panels. Requirements of Section 10.7 for deep beams apply regardless of whether the member is prestressed or nonprestressed.

Shear—Chapter 11 for both prestressed and nonprestressed panels.

Bearing—Sections 10.14 and 15.8.

Combined bending and axial load—Section 10.3.

3.2.4 Combined bending and axial load

3.2.4.1 General—All forces listed in Section 3.2.2 should be considered in designing wall panels for combined bending and axial load. Also, the effects of secondary forces caused by deflection, variable moment of inertia, stiffness, and duration of load should be considered. Axial forces, bending moments, and shear forces should be determined from an analysis of the structure. Considerations of member translation, joint translation, or both, should be considered in the analysis. In place of the aforementioned procedure, compression member design may be based on the approximate procedures given in Section 3.2.4.2.

3.2.4.2 Approximate evaluation of slenderness effect—Procedures described in Section 10.10 of ACI 318-08 should be followed for determining the unsupported length, effective length, and radius of gyration of precast wall panels.

(a) Slenderness effects may be neglected if slenderness conforms to Section 10.10.4.1 or 10.10.4.2 of ACI 318-08. For compression members with slenderness kl_u/r greater than 150, an analysis according to Section 3.2.4.1 of this guide should be made.

(b) The magnified moment for design of a compression member should be determined according to Section 10.10.5.1 of ACI 318-08.

(c) For precast wall panels considered to be reinforced concrete compression members by these recommendations, the provisions of Section 10.10.5.2 of ACI 318-08 can be used instead of a more accurate analysis.

(d) For precast wall panels considered to be prestressed concrete compression members by these recommendations, the provisions of Section 5.9 of the *PCI Design Handbook* (MNL-120-10) can be used instead of a more accurate analysis.

(e) An equivalent uniform bending moment factor, defined in accordance with Section 10.10 of ACI 318-08 should be considered for precast wall panels braced against sidesway and without transverse load between supports.

(f) The minimum eccentricity, according to Sections 10.3.6 and 10.3.7 of ACI 318-08, as appropriate, should be considered for precast wall panels when no bending moment occurs at either end of the panel.

3.2.5 Reinforcement—Precast wall panels are not required to have lateral hoop or spiral reinforcement unless analysis indicates this reinforcement is required.

Limits of reinforcement for precast wall panels should conform to Sections 7.10, 7.12, 10.9, 14.3, and 18.8 of ACI 318-08. Two-way reinforcement is not required for some essentially one-way panels, such as hollow-core panels.

3.3—Effective dimensions

3.3.1 Effective thickness

3.3.1.1 General—The effective panel thickness for design may be different from the total panel thickness. The following sections explain how to determine the effective thickness for design purposes and Fig. 3.3.1.1(a) through 3.3.1.1(c) provide the general characteristics of the various effective thicknesses.

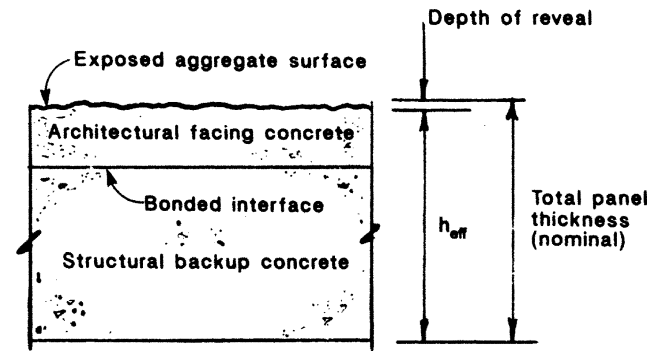
3.3.1.2 Architectural faced panels—The effective thickness of a wall panel with an integral exposed aggregate surface should be determined by subtracting the depth of aggregate reveal from the total panel thickness if the depth of aggregate reveal exceeds 3 percent of the total thickness. The effective thickness of a wall panel with a noncomposite facing should not include the separate facing thickness.

3.3.1.3 Solid, hollow-core, and ribbed panels—Effective panel thickness should be determined by Eq. (3.3.1.3).

$$h_{eff} = \sqrt[3]{\frac{12I_g}{b}} \quad (3.3.1.3)$$

where I_g is the uncracked moment of inertia accounting for voids or ribs, if they exist.

3.3.1.4 Sandwich panels—Effective thickness of a sandwich panel may be assumed equivalent to the effective thickness of the two wythes plus insulation only if mechanical shear connectors capable of developing full composite action are used to connect the interior and exterior wythes. In such cases, the effective thickness may be determined from Eq. (3.3.1.3).

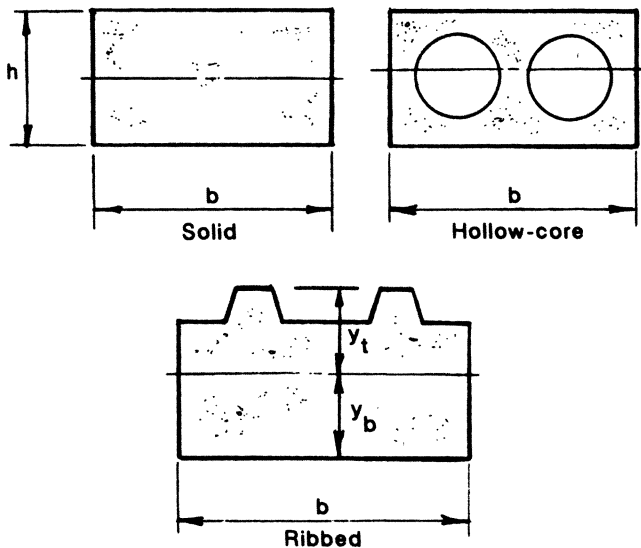


$h_{eff} = \text{Total panel thickness} - \text{depth of reveal}$
(if depth of reveal exceeds 3% of nominal thickness)

or

$h_{eff} = \text{Total panel thickness}$

Fig. 3.3.1.1(a)—Effective thickness of architectural faced panels.



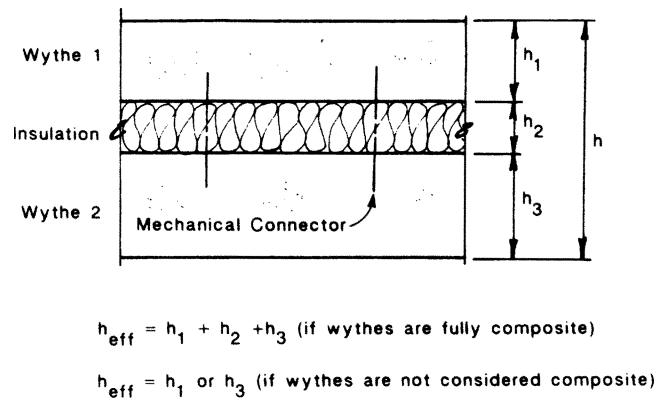
$I_g = \text{Uncracked moment of inertia}$

$$h_{eff} = \sqrt[3]{\frac{12 I_g}{b}}$$

Fig. 3.3.1.1(b)—Effective thickness of solid, hollow-core, or ribbed panels.

If the insulation core is cellular lightweight concrete or lightweight concrete made with mineral aggregates, the shear transfer through the insulation core should not exceed the shear allowed by the strength of the insulating concrete core.

When only partial composite action between wythes exists, and loadings are from lateral forces or long-term sustained loads, the two wythes should be considered as separate members unless testing is conducted to verify panel behavior. Refer to Section 3.4.2 for limitations on the maximum slenderness ratio of the load-bearing wythe.



$$h_{eff} = h_1 + h_2 + h_3 \text{ (if wythes are fully composite)}$$

$$h_{eff} = h_1 \text{ or } h_3 \text{ (if wythes are not considered composite)}$$

Fig. 3.3.1.1(c)—Effective thickness of sandwich panels.

3.3.1.5 Panels of irregular shape—Panels not conforming to the configurations listed in this section may have the effective thickness determined by analysis or testing based on cracking moment.

3.3.2 Effective width—When concentrated loads or bending moments are applied to the top and bottom of a wall panel, the effect of local stress near the applied concentrated load or bending moment should be investigated. The effective width for a concentrated load may not exceed the center-to-center distance between supports, or the width of the loaded portion plus six times the wall panel effective thickness on each side of the concentrated load. The effective width for concentrated bending moments may not exceed the effective thickness of the wall panel or corbel width at the point of concentrated bending moment, whichever is greater, plus three times the effective wall panel thickness each side of the concentrated bending moment. Effective width may also be determined by a more accurate analysis.

3.4—Limiting dimensions

3.4.1 General—Limiting dimensions for precast wall panels should be based on requirements of concrete placement, protection of prestressed and nonprestressed reinforcement, fire resistance, member and local stability, deflection, handling, transportation, and concrete cracking.

3.4.2 Distance between supports—Spacing of lateral supports for a precast wall panel loaded in flexure only should not exceed 50 times the effective width of the compression flange or face. The maximum slenderness kl_u/r of a precast wall panel should not exceed 200. Spacing between lateral supports of a precast panel carrying axial load and bending moment should not exceed 50 times the effective width of the compression face or flange for stemmed sections. Lateral bracing should be attached to the compression region of the member cross section needing lateral support unless it can be shown that other portions of the cross section have sufficient stiffness to brace the member.

3.5—Serviceability

3.5.1 General—The action of service loads on deflections perpendicular and parallel to the wall and should be considered. Fatigue, impact, cracking, and in-plane lateral

Table 3.5.2—Deflection limits for precast wall panels

Member type	Deflection to be considered	Deflection limitation
Load-bearing precast wall panels	Immediate deflection due to combined effects of prestress (if any), self-weight, and superimposed dead load.	$l/240$ but not greater than 3/4 in. (20 mm)
	Immediate deflection due to live load, in. (mm).	$l/360$ but not greater than 3/4 in. (20 mm)
Non-load-bearing precast wall panel elements likely to be damaged by large deflection	That part of the total deflection after the installation of the non-load-bearing element (sum of long-time deflection due to all sustained loads and the immediate deflection due to live load).	$l/480$ but not greater than 3/4 in. (20 mm)

stability at service load conditions should be accounted for in design.

3.5.2 Computed permissible deflections—Precast wall panel dimensions should be chosen so that under service load conditions, deflection of any point on the panel measured from its original position should not exceed the limits given in Table 3.5.2. In calculating deflection, nonlinear behavior of the materials, structural member, or both, should be recognized.

3.5.3 Cracking

3.5.3.1 Acceptability of cracking—Precast wall panels typically undergo less cracking than cast-in-place concrete, but they are not generally crack-free. Computations based on current engineering practice assume that cracks will occur in a concrete member even though they may not be visible to the naked eye. It is the control and acceptability of these cracks that should be evaluated. Wall panels containing cracks up to 0.012 in. (0.30 mm) wide should be acceptable. Crack size limitations are specified for structural reasons. Additional guidance on cracking and its causes can be found in the PCI MNL-116-99, PCI MNL-117-96, PCI MNL-120-10, PCI MNL-122-07, ACI 224.1R-07, and ACI 224R-01.

Aesthetic crack size limitation depends on surface texture and required appearance. On coarse textured surfaces, such as exposed aggregate concrete, and on smooth surfaces comparable to the best cast-in-place structural concrete, the structural limitation would be aesthetically acceptable. Note also that cracks will become more pronounced on surfaces receiving a sandblasted or acid-etched finish.

Cracks in precast concrete panels may be classified as hairline, cleavage, or fracture cracks.

- Hairline cracks are surface cracks of minute width, visible but not measurable without magnification.
- Cleavage cracks are cracks not over 0.012 in. (0.30 mm) wide that, in the judgment of the inspector, penetrate at least to the plane of the nearest reinforcing steel.
- Fractures are total cleavages of measurable width through which water may pass freely.

Crazing consists of hairline cracks in an approximate hexagonal or octagonal pattern on the concrete surface. These can occur in many panels, but they are not readily visible in exposed aggregate surfaces or when the concrete is dark. They are most apparent on white panels, flat surfaces, and smooth finishes. Crazing is of little structural importance and should not be cause for rejection. If panels are to be installed in an environment that may be the source of considerable soiling, it may be advisable to avoid smooth concrete finishes to render potential crazing less visible.

3.5.3.2 Crack prevention and control—Significant reductions in crack widths can be obtained by proper selection and location of reinforcement. Maintaining accurate steel positioning during the casting operation also aids in reducing crack widths. Reinforcement is most effective when it consists of more closely-spaced, smaller-diameter bars or wire, particularly in thin sections. Welded wire fabric reinforcement is commonly used instead of reinforcing bars because of the relatively close wire spacing (4 to 6 in. [100 to 150 mm] or less).

Flexural reinforcement distribution requirements in Section 10.6 of ACI 318-08 should be followed for reinforced precast or architectural wall panel surfaces not exposed to view. When the geometry of the precast member is more like that of a two-way slab, flexural reinforcement requirements of Section 10.6 of ACI 318-08, may lead to crack widths wider than expected. For information on crack repair, refer to ACI 224.1R-07.

3.5.3.3 Limit on flexural tension—For conventionally reinforced and prestressed wall panels where the exposed surface is to remain free of discernible cracks, the maximum flexural tension in the member under loads produced by stripping, handling, transportation, impact, and live load

effects should be less than $\sqrt[3]{f'_c}$. The tensile strength value of concrete should be modified according to Section 11.2 of ACI 318-08 if lightweight aggregate concrete is used.

3.6—Connections

3.6.1 General—Wall panel units should be safely and adequately seated and anchored by mechanical means capable of sustaining all loads and stresses that may be applied to the wall panel, including positive or negative wind pressures and seismic forces where required by ACI 318-08. Whenever possible, panels should be concentrically supported to avoid bowing and warping due to stress differential between inside and outside faces of the panel.

When the wall panel is designed to serve as a structural member, it may be required to carry imposed vertical loads, resist bending and shear (other than that caused by its own weight and volumetric changes), or it may be designed to function as a shear wall. When wall panels are designed to transmit load from one to another, consideration should be given to the additional loads required for design of the connection(s). Concepts for design of connections for precast wall panels may be found in PCI MNL-117-96 and PCI MNL-123-88.

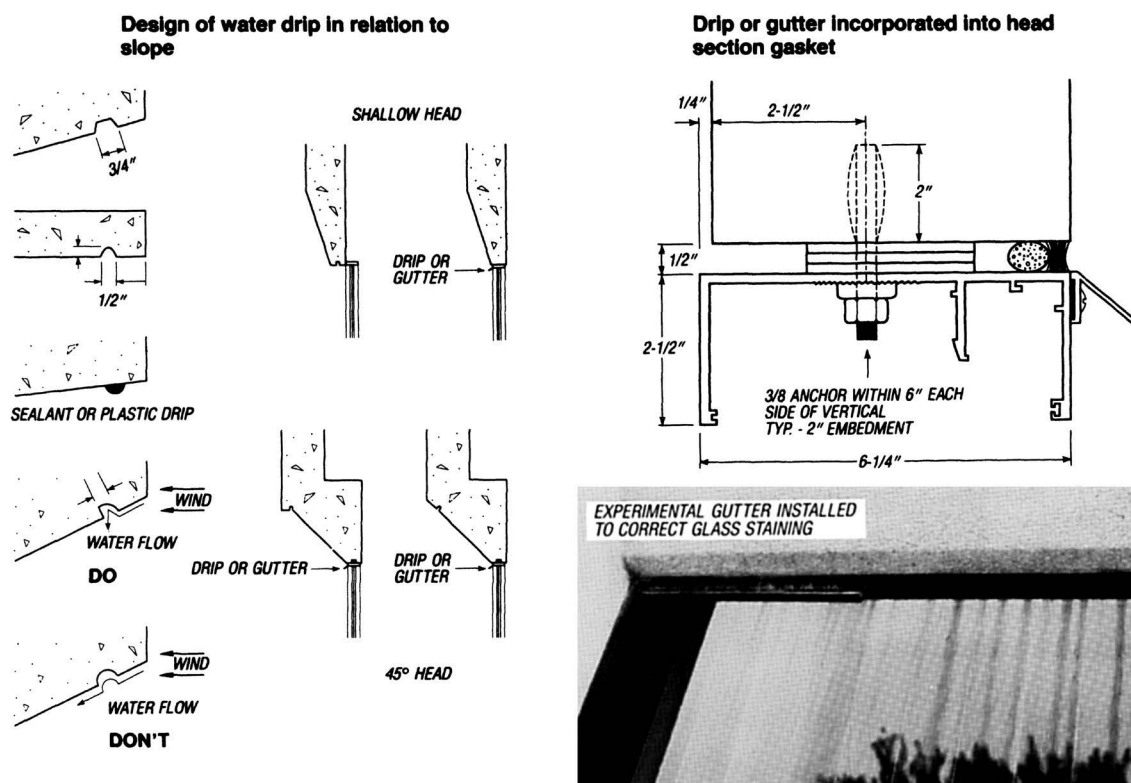


Fig. 3.7.1—Design of water drip in relation to slope.

3.6.2 Panel movement—Wall panel connection assemblies should be designed to allow for panel movement induced by temperature or caused by volumetric change in the concrete, moisture differential, creep in prestressed panels, and differential movement or drift between building frame and wall panel units. Design guidance design for these conditions can be found in PCI MNL-120-10 and Gaylord et al. (1996).

3.6.3 Bearing seats—Because of the indeterminacy in the analysis of load-transfer connection assemblies, bearing seats should be provided for panels weighing more than 5000 lb (2268 kg). These connections are typically corrosion protected and behind a weather proof joint; hence, maintenance is not required.

The designer should avoid hanging panels from inserts, anchors, or other connection devices in direct tension near the top edge of the panel. Clips, clamps, welding plates, and brackets are commonly used to resist horizontal and lateral loads. When they are intended to transfer the panel weight to the structure, rigorous analysis is required in their design, and special precautions should be enforced to ensure proper installation.

3.6.4 Haunches—Concrete haunches used to positively seat panels should conform to shear requirements of Section 11.9 of ACI 318-08 and should be designed for eccentric loading and combined shear, bending, tension, and bearing stresses. The effect of eccentricity, which causes panel deflection, should be considered in the design of panel reinforcement.

3.6.5 Panel inserts—The design of wall panel inserts that are part of a connection assembly should be based on the

load factors and strength reduction factors (ϕ factors) specified in Sections D.4.4 and D.4.5 of ACI 318-08. Connections should not be the weak link in a precast system. Inserts should have a safety factor consistent with the insert manufacturer's recommendation.

3.6.6 Fire resistance—Wall panel connections should be fireproofed to have minimum fire resistance equivalent to that required by local code for the wall panels.

3.6.7 Weld design—Potential relative movement between the panel and supporting structural frame or adjacent panels should be investigated when designing welds. The effects of possible concrete cracking due to welding heat on the precast panel or its supporting concrete frame should be considered in the design of the connection assembly.

3.7—Architectural features

3.7.1 Drip details—Directed slow-water runoff of rain-water over building facades and dirt accumulation can contribute to staining or etching of glass surfaces. This phenomenon is fully explained in PCI MNL-122-07. Appropriate building details can reduce the water discharged to the glass. Concrete frames at window heads should, wherever possible, be designed so that they do not splay down and back toward the glass unless drip details are incorporated into the frames. Without drip details, a direct, slow wash down of the glass should be anticipated.

The drip section should be designed in relation to the slope of the concrete surface (Fig. 3.7.1). To avoid a weakened section that is likely to chip, the drip should not be located too close to the edge of the precast unit. Using edge

drips and a second drip or gutter serves as a dual line of defense against slow water runoff. This can be accomplished by having a cast-in drip in the panel or by using of aluminum or neoprene extrusions across the window head, which have either an integral gutter or an extended drip lip of at least 1 in. (25 mm) (Fig. 3.7.1).

3.7.2 Joint size and location—Joints between precast panels or panels and adjacent building materials should be wide enough to accommodate anticipated panel and building movements. No joint should ever be designed to be less than $3/8 \times 3/8$ in. (10 x 10 mm). Particular care should be given to joint tolerances so the joint sealant system performs within its design capacities. For optimum performance and maximum sealant life, recommendations of the sealant manufacturer should be followed.

Panels less than 15 ft (4.6 m) long may have 1/2 in. (13 mm) joints, but all other panels should have at least 3/4 in. (19 mm) joints. Corner joints should be at least 1/4 in. (6 mm) to accommodate extra movement and bowing that occurs there. Joint widths less than or equal to 3/8 in. (10 mm) are not recommended for any sealant installation. When joints are too narrow, adjacent panels or building materials may contact each other and be subjected to induced loading, distortion, cracking, and crushing of ends.

CHAPTER 4—TOLERANCES

4.1—General

Precast structures should be designed and detailed so that the complete structure will be safe, functional, aesthetically appealing, and economical. No structure is exactly level, plumb, and straight, so all construction and materials should be specified with tolerances limiting deviation from design values. These tolerances require monitoring to construct the structure as designed. General construction tolerances for cast-in-place and precast concrete have been summarized in ACI 117-10, ITG-7-09, and PCI MNL-135-11. This chapter presents tolerances specifically applicable to precast concrete wall panels. Three tolerance groups should be established as part of precast concrete wall panel design. Wall panels and their component details should conform to:

- Product tolerances (Section 4.4).
- Installation tolerances (Section 4.6).
- Interfacing tolerances (Section 4.7).

When tolerances are understood and provided for in the design stage, determining and specifying them is easier. The precaster, constructor, and erector should understand these allowances to construct the structure as designed.

4.2—Reasons for tolerances

Tolerances are needed for product, installation, and interfacing for the following reasons:

- *Structural considerations*—to ensure structural design accounts for factors sensitive to variations in dimensional control. Examples include eccentric loading, bearing areas, hardware and hardware anchorage locations, and locations of reinforcing or prestressing steel.
- *Performance*—to ensure acceptable performance of joints and interfacing materials in the finished structure.

- *Appearance*—to ensure that deviation from theoretical requirements will be controllable and result in acceptable appearance. Large deviations are objectionable, whether they occur in an isolated manner or cumulatively.
- *Cost*—to ensure ease and speed of fabrication and installation by having a known degree of accuracy in the dimensions of the precast members.
- *Legal considerations*—to avoid encroaching on building lines.
- *Contractual*—to establish a known acceptability range and to establish responsibility for developing, achieving, and maintaining mutually agreed-on tolerance values.

4.3—Role of engineer/architect

The engineer/architect should coordinate the tolerances for precast work with the requirements of other trades whose work relies on or is adjacent to the precast. Tolerances should be reasonable, realistic, and within generally accepted limits, because fabrication and installation costs are directly related to degree of precision required. Thus, it is economically desirable and practically safer to design with maximum flexibility and to maintain liberal tolerance requirements. Tolerances in this guide are basic guidelines only. The engineer/architect determines whether a deviation from the allowable tolerances affects safety, appearance, or other trades.

When design involves features sensitive to the cumulative effect of tolerances on individual portions, the engineer/architect should anticipate and provide for this effect by setting a cumulative tolerance limit or by providing escape areas where accumulated tolerances or fabrication errors can be absorbed. All tolerance consequences for a particular design should be investigated to determine whether a change is necessary in the design or in the tolerance level for the design. There should be no possibility of minus tolerances accumulating so that the bearing length of members is reduced below the required design minimum. In this case, the engineer/architect should specify minimum bearing dimensions.

Careful inspection of listed tolerances reveals that many times one tolerance overrides another. The permitted variation for one element of the structure should not require another element of the structure to exceed its tolerances. The precaster and general contractor should review restrictive tolerances to determine compatibility with other elements and that the tolerances can be met. For example, a requirement stating “no bowing, warping, or movement is permitted” is impractical. All involved in the design and construction process should understand that tolerances herein are for guidance on the range of acceptability and not an automatic standard for rejection. If these tolerances are exceeded, the engineer/architect may accept the product if it meets any of the following criteria:

- (a) Exceeding the tolerance does not affect structural integrity, architectural performance, or serviceability requirements of the member.

(b) Member can be brought within tolerance by structurally and architecturally satisfactory means.

(c) Total erected assembly can be modified to meet all structural and architectural requirements.

4.4—Product tolerances for wall panels

4.4.1 General—Product tolerances cover the dimensions and dimensional relationships of individual precast concrete members. All tolerances should be based on a practical degree of accuracy, which is achievable while satisfying functional and appearance requirements and preventing costs from becoming prohibitive. This requires consideration of the amount of repetition, the size, and other characteristics of the precast member.

Fabrication tolerances are standardized throughout the precast industry and for economic reasons should be made more demanding only where absolutely necessary. For example, bowing or warping tolerances for flat concrete panel members with a honed or polished finish might have to be decreased to 50% of typical tolerances to avoid joint shadows. When design details lead to an alignment problem or provide inadequate joint size, product tolerance may need adjustment to compensate for joint design problems. In establishing casting tolerances for panels, the following items should be considered:

- Length or width dimensions and straightness of the precast element will affect the joint dimension, dimensions of openings between panels, and perhaps the overall structure length.
- Out-of-square panels can cause tapered joints and make adjustment of adjacent panels extremely difficult. Sealant application difficulties due to tapered joints can lead to water leakage problems.
- Thickness variation of the precast unit becomes critical when interior surfaces are exposed to view. A nonuniform thickness of adjacent panels will cause offsets at the front or rear faces of the panels.

4.4.2 Dimensional tolerances—Architectural precast concrete panels should be manufactured and installed so that each panel face exposed to view after installation complies with the dimensional requirements shown in ITG-7-09. Tables 2.1 and 4.1 in ITG-7-09 also list the position tolerance for cast-in items within the panel. These are for typical generic panels, and the tolerances may require adjustment for specific job conditions. Cast-in grooves, reglets, or lugs that are to receive glazing gaskets should be held close to their correct location. Gasket manufacturers place restrictive tolerances on groove width and surface smoothness necessary to obtain a proper moisture seal of the gasket. Misalignment of reglets at corners, or casting them in a warped or racked position restricts proper installation of the glazing gasket.

Dimension tolerances for standard precast ribbed panels and hollow-core slabs used as wall panels are shown in Table 2.1 of ITG-7-09. Standardized ribbed and hollow-core members, typically used for roof and floor units, are also frequently used as wall panels. Tolerances for these standardized structural units are generally more liberal than those for architectural panels. If the engineer/architect cannot accept

the standard structural tolerances of ribbed and hollow-core units when using them as wall panels, they should specify the architectural tolerances as given in Table 4.1 of ITG-7-09 or other tolerances as required.

4.4.3 Warping and bowing—Warping and bowing are defined in Section 2.2 and illustrated in Fig. 1.2 and 1.5 of ITG-7-09. Nonsymmetrical placement of reinforcement may allow warping due to lack of restraint of drying shrinkage and thermal movements. Note that surface out-of-planeness (also defined in Section 2.2) is differentiated from bowing because it is not a characteristic of the entire panel shape, but a local smoothness variation. Tolerance for local smoothness is checked with a straightedge or the equivalent as shown in Fig. 1.4 of ITG-7-09 and the measurement of warping in Fig. 1.5 of ITG-7-09.

Table 4.4.3 shows a relationship between overall flat panel dimensions and cross-sectional thickness. If the thickness is less than suggested in Table 4.4.3, warping tolerances should be reviewed for the possibility of increasing the tolerance. Panels thinner than those in Table 4.4.3, ribbed panels, and panels manufactured using aggregates larger than 3/4 in. (19 mm) exposed at the surface need further consideration when establishing bowing and warping tolerances. Tolerances for flat panels of nonhomogeneous materials, such as two widely different concrete mixtures or natural stone veneer with concrete backup, should be reviewed as they may have to be increased or reduced to meet design criteria.

4.5—Installation tolerances for wall panels

4.5.1 Discussion—Installation tolerances are required for the functional matching of the precast elements with the building structure. The engineer/architect should set tolerances that are compatible with the desired architectural expression and detailed to the building structure and site conditions. Installation tolerances should be set to achieve uniform joint and plane wall conditions considering the individual element design, shape, thickness, composition of materials, and overall scale of the element in relation to the building. Installation tolerances affect the work of several trades and should be consistent with the tolerances specified for those trades. It is the responsibility of the engineer/architect to ensure that tolerances are compatible.

The engineer/architect should review proposed installation tolerances with the panel manufacturer and the erector before installation commences. Proposed changes by the manufacturer or erector from the original plan should be stated in writing and noted on installation drawings. Agreement should be reached before scheduling equipment for panel installation.

The general contractor, in consultation with the precast concrete installation contractor, should check dimensions and location of the in-place structure before placing precast panels. Any dimensional discrepancies that may affect installation should be reviewed and resolved with the engineer/architect before starting installation. The contractor may have to make corrections to the interfacing structure.

Location or installation tolerances for wall panels should be noncumulative. Recommended tolerances are listed in

Table 4.4.3—Guidelines for panel thicknesses for overall panel stiffness consistent with suggested normal panel bowing and warping tolerances

Panel dimensions	8 ft (2.44 m)	10 ft (3 m)	12 ft (3.66 m)	16 ft (4.88 m)	20 ft (6.10 m)	24 ft (7.32 m)	28 ft (8.53 m)	32 ft (9.75 m)
4 ft (1.22 m)	3 in. (76.2 mm)	4 in. (102 mm)	4 in. (102 mm)	5 in. (127 mm)	5 in. (127 mm)	6 in. (152 mm)	6 in. (152 mm)	7 in. (178 mm)
6 ft (1.83 m)	3 in. (76.2 mm)	4 in. (102 mm)	4 in. (102 mm)	5 in. (127 mm)	6 in. (152 mm)	6 in. (152 mm)	6 in. (152 mm)	7 in. (178 mm)
8 ft. (2.44 m)	4 in. (102 mm)	5 in. (127 mm)	5 in. (127 mm)	6 in. (152 mm)	6 in. (152 mm)	7 in. (178 mm)	7 in. (178 mm)	8 in. (203 mm)
10 ft (3 m)	5 in. (127 mm)	5 in. (127 mm)	6 in. (152 mm)	6 in. (152 mm)	7 in. (178 mm)	7 in. (178 mm)	8 in. (203 mm)	8 in. (203 mm)

Table 5.1 of ITG-7-09, which shows installation tolerances for nonstructural architectural wall panels, as well as Table 3.1 of ITG-7-09, which shows installation tolerances for structural wall panels.

4.5.2 Control points and benchmarks—To ensure accurate application of installation tolerances, the general contractor should establish and maintain accurate control points and benchmarks in areas that will remain undisturbed until final completion and acceptance of a project.

The general contractor should provide the erector with a building perimeter offset line at each floor approximately 2 ft (0.6 m) from the edge of the floor slab and benchmarks on all perimeter columns. Offset lines and benchmarks should be maintained until final completion and acceptance of the work. They may be scored into columns and floor slabs or set as chalk lines and lacquered for protection.

4.5.3 Joint problems—Width variations between adjacent joints can be minimized by setting out joint centerlines equally spaced along an elevation and centering panels between them. The larger the panels, the wider the joint should be to accommodate realistic tolerances in straightness of panel edge, slope of edge, and panel width. Alignment for exterior elements should be controlled by assuming that the outside face of the element is critical. Variations from true length or width dimensions of the overall structure are normally accommodated in the joints. Where this is not feasible or desirable, variations should be accommodated at the corner elements, in expansion joints, or in joints adjacent to other wall materials.

Joint widths should be designed as liberally as practicable if variations in overall building dimensions are to be absorbed in the joints. This may be coupled with a closer tolerance for variations from one joint to the next for appearance purposes. Individual joint width tolerance should relate to the number of joints over a given building dimension. For example, to accommodate reasonable variations in actual site dimensions, a 3/4 in. (19 mm) joint may be specified with a tolerance of $\pm 1/4$ in. (6 mm) but with only a 3/16 in. (4.5 mm) differential allowed between joint widths on any one floor, or between adjacent floors.

Where a joint has to match an architectural feature such as false joints, a $\pm 1/4$ in. (6 mm) variation from the theoretical joint width may not be acceptable and a tighter tolerance specified. Adjustment in building length will then have to be

accommodated at the corner panels or in joints adjacent to other wall material.

If reasonable tolerances and adjustments are designed into the construction details and adhered to, the erector should be able to:

- Minimize joint irregularities such as tapered joints (panel edges not parallel).
- Minimize jogs at intersections.
- Minimize non-uniform joint widths.
- Maintain proper opening dimensions.
- Properly construct all precast connections.
- Align vertical faces of the units to avoid offsets.
- Prevent accumulation of tolerances.

More precise installation and general appearance improvements are thus achieved.

4.6—Interfacing considerations

4.6.1 General—Interface tolerances and clearances are those required for joining different materials and accommodating the relative movements between such materials during the life of the building. They cover products installed before precast installation and after the precast members are in place. The engineer/architect should provide for proper clearances between the theoretical face of the structure and the back face of the precast element. The face of the structure may be precast concrete, cast-in-place concrete, masonry, or a structural steel frame. Adjacent materials include products such as glass or subframes that are installed after precast panels are in place. Clearance space provides a buffer where installation, product, and interface tolerances can be absorbed.

Where matching of the manufactured materials depends on work at the construction site, interface tolerances should equal installation tolerances. Where execution is independent of site work, tolerances should closely match the standard tolerances of the materials to be joined. Fabrication and installation tolerances of other materials should be considered in design. Precast elements should be coordinated with and accommodate the other structural and functional elements comprising the total structure. Unusual requirements or allowances for interfacing should be stated in the contract documents.

4.6.2 Building frame tolerances—Installation tolerances for precast panels are largely determined by the alignment and dimensional accuracy of the building foundation.

Table 4.6.2—Supplementary tolerances for cast-in-place concrete frames to which precast concrete is to be attached

Footings, caisson caps, and pile caps	
Variation of bearing of surface from specified elevation	±1/2 in. (±13mm)
Piers, columns, and walls	
a. Variation in plan from straight lines parallel to specified linear building lines:	1/40 in./ft (0.7 mm/0.3 m) for adjacent members less than 20 ft (6 m) apart or any wall or bay length less than 20 ft.
	1/2 in. (13 mm) for adjacent members 20 ft (6 m) or more apart or any wall or bay length 20 ft (6 m) or more.
b. Variation in elevation from lines parallel to specified grade lines:	1/40 in./ft (0.7 mm/0.3 m) for adjacent members less than 20 ft (6 m) apart or any wall or bay length less than 20 ft.
	1/2 in. (13 mm) for adjacent members 20 ft (6 m) or more apart or any wall or bay length 20 ft (6 m) or more.
Anchor bolts	
a. Variation from specified location in plan:	3/4 in. (19 mm) and 7/8 in. (22 mm) bolts ±1/4 in. (6 mm)
	1 in. (25 mm), 1-1/4 in. (32 mm), and 1-1/2 in. (38 mm) bolts ± 3/8 in. (10 mm)
	1-3/4 in. (44 mm), 2 in. (51 mm), and 2-1/2 in. (64 mm) bolts ±1/2 in. (13 mm)
b. Variation center-to-center of any two bolts with an anchor bolt group:	≤1/8 in. (≤3 mm)
c. Variation from specified elevation:	±1/2 in. (±13 mm)
d. Anchor bolt projection:	-1/4 in., +1/2 in. (-6 mm, +13 mm)
e. Plumbness of anchor bolt projection:	±1/16 in./ft (±1.6 mm/0.3 m)

tion and frame. The general contractor is responsible for the plumbness, levelness, and alignment of the foundation and structural frame, including the location of all bearing surfaces and anchorages for precast products. Controlling foundation and building frame tolerances is of critical importance and these tolerances should be included in the contract documents. To install precast elements plumb, square, and true, the actual location of surfaces affecting the alignment of precast elements should be known before installation. The levels of floor slabs and beams, vertical alignment of floor slab edges, and the plumbness of columns or walls should be considered.

Concrete cast-in-place frames to which precast elements are attached should meet the tolerances in Table 4.6.2 and the requirements of ACI 301-10 and ACI 117-10.

Variations in floor heights are greater and more prevalent in cast-in-place structures than in other structural frames. This affects the alignment of precast insert with the cast-in-place connection device.

Tolerances for cast-in-place structures may have to be increased to reflect local trade practices, structure complexity, and environmental conditions. In Fig. 3.3 of ITG-7-09, installation tolerances for beams and spandrels are shown, specifically precast element to precast element, precast to cast-in-place concrete and masonry, and precast to steel frame.

4.6.3 Mixed construction—ACI 117-10 applies only to reinforced concrete and masonry buildings, and AISC 325-05 only to steel building frames. Tolerances in these standards do not apply to buildings of mixed construction, such as concrete floor slabs carried by steel beams or concrete-encased structural steel members. Obviously, the location of the face of the concrete on an encased steel member and the

location of the steel member itself are both critical. Because the alignment of mixed construction members and encased members is not controlled by referencing the aforementioned standards, the engineer/architect should require that the location of all such materials contiguous to the precast unit be controlled within stated limits. One recommendation is that tolerances be no more than those specified in ACI 301-10 for reinforced concrete buildings. When there is doubt as to the level of mixed construction tolerances, the precast concrete manufacturer may be consulted for advice.

4.6.4 Steel building frames—Precast concrete panels should be erected as uniformly as possible around the entire perimeter of the structure to avoid pulling the steel framing out of alignment. Steel building frames have different tolerances from those discussed previously. Tolerances for steel frame structures make it impractical to maintain precast concrete panels in a true vertical plane. Based on the allowable steel frame variations, it would be necessary to provide for a 3 in. (75 mm) adjustment in connections up to the twentieth story and a 5 in. (126 mm) adjustment in connections above the twentieth story if the engineer/architect insists on a true vertical plane. Connection adjustments of this magnitude are not economically feasible. Therefore, the precast concrete wall should follow the steel frame.

In determining tolerances, attention should be given to possible deflections and rotation of structural members supporting precast concrete. This is particularly important for bearing on slender or cantilevered structural members. If frame deflection is sensitive to location or eccentricity of the connection, tolerances for location or eccentricity should be given.

Consideration should be given to initial and long-term deflections caused by creep of the supporting structural

members. Beam and column locations should be uniform in relation to the precast units with a constant clear distance between precast concrete and support elements.

A structural steel frame presents installation and connection problems different from that of a concrete building frame. Structural steel cross sections are relatively weak in torsion compared with concrete cross sections. Structural steel members generally require that the load be applied directly over the web or that the connection be capable of supporting the induced torsional moment. This can require a stronger connection and create installation problems when rolling tolerances of the steel beam approach their limits. When detailing precast elements for attachment to steel structures, allowance should be made in the precast connections, such as sliding connections and oversized holes; for sway in tall, slender steel structures with uneven loading; and for deflections due to thermal effects.

Designs should provide for vertical adjustment of precast concrete panels supported by the steel frame. An accumulation of axial shortening of axially loaded steel columns will result in the unstressed panels supported at each floor level being higher than the steel frame connections to which they should be attached. Sometimes, non-load-bearing precast elements become load-bearing even though the design does not allow for load. This can result in cracking.

The clearance necessary for wall installation depends on wall design, dimensional accuracy of the building frame or other construction to which the wall is connected, and the adjustment limits permitted by connection details. When connections to the face of spandrel beams or to columns are required, more clearance will be needed to install fasteners than when anchors are located on the sides of columns and the top, bottom, or both faces of beams.

4.7—Clearances and tolerances for constructibility

4.7.1 Suggested minimum clearances—Clearance, or interface space between members, should be specified to facilitate construction. Suggested minimum clearances are:

- Between precast panels and adjacent precast members—1/2 in. (13 mm).
- Between precast panels and cast-in-place concrete—1 in. (25 mm), but 1-1/2 in. (38 mm) is preferred.
- Between precast panels and steel frame—1 in. (25 mm).
- Between precast panels and the frame of tall irregular structures—2 in. (50 mm).
- Between precast column cladding and column—1-1/2 in. (38 mm), but 3 in. (75 mm) is preferred.

If clearances are practically assessed, they will solve many tolerance problems. The nominal clearance dimension shown on the installation drawings should equal the actual clearance required plus the outward tolerance permitted for the adjacent structural frame. Nominal clearances should be determined on the assumption that the structural frame will be as far out of position in the wrong direction as allowed. Connections should be designed to accommodate clearance plus inward tolerance.

4.7.2 Connection problems—Connections should have the maximum adjustability that is structurally or architecturally feasible. Closer tolerances are required for bolted connections than for grouted connections. Connections should provide for vertical, horizontal, and lateral adjustments of 1 in. (25 mm) minimum to accommodate any misalignment of the support system and the precast elements. Location of hardware items cast into or fastened to the structure by the general contractor, steel fabricator, or other trades should be determined with specified tolerances for all site placement. Unless another value is specified by the engineer/architect, tolerances for such locating dimensions should be +1 in. (25 mm) in all vertical and horizontal directions plus a slope deviation of no more than +1/4 in. (6 mm) for the levelness of critical bearing surfaces.

Connection details should provide for the possibility that bearing surfaces are misaligned or warped from the desired plane. Adjustments can be provided by the use of dry-pack concrete, nonshrink grout, or elastomeric pads if the misalignment from the horizontal plane does not exceed 2 in. (50 mm).

When possible, connections should be dimensioned to the nearest 1/2 in. (13 mm). Minimum clearance between parts within a connection should not be less than 1/4 in. (6 mm), with 1/2 in. (13 mm) preferred. Minimum clearance or shim space between various connection elements should be 1 in. (25 mm).

Where a unit is not erected within the tolerances assumed in the connection design, the structural adequacy of the installation should be checked and the connection design should be modified if tolerances are exceeded. No element should be left in an unsafe support condition. Adjustments in the prescribed tolerances should be made only after approval by the engineer/architect.

CHAPTER 5—MATERIALS

5.1—Introduction

Basic materials used in fabrication and installation of precast concrete wall panels are the same as those used in cast-in-place structural concrete, but precast concrete wall panels also make use of special materials, casting techniques, and curing. Exposed aggregates, admixtures, inserts, and specialty coatings are used to enhance aesthetic appearance. This chapter describes the following materials as used in precast concrete panel construction.

- Portland cement.
- Standard and decorative aggregates for exposed facing.
- Admixtures.
- Insulating materials and wythe connectors.
- Reinforcement and inserts.
- Curing materials and sealers.
- Joint sealants and fillers.
- Surface retarders.
- Form release agents.
- Thin brick.

Most of these materials are considered in more detail by other ACI committees.



5.2—Portland cement

5.2.1 General—Usual industry practice is to use white, buff, or gray portland cement or a mixture of white and gray cements meeting ASTM C150/C150M-11 requirements for Type I or Type III. White cement usage should be clearly specified when it is required. Cement Types II, IV, and V are seldom used in precast panels. When using any special cement, take every precaution to ensure that early concrete strengths are adequate.

5.2.2 Single source—For any given project, enough cement for the entire project should be procured from a single source so all cement is the same brand and type. Some precasters prefer to obtain a single, one-time, one-batch shipment for a given project to minimize color variations. Elimination of color variation is not possible because variables in other materials and in panel fabrication can have effects.

5.2.3 Storage—Dry, covered storage areas should be provided for bulk or bagged cement. Bagged cement should be stored on wooden pallets and out of contact with tanks or outer storage building walls where condensation could occur. To avoid pack set, bags should not be stored more than two pallets high, or 7 ft (2.1 m) total height.

5.2.4 Sampling—In case of future problems with strength or color uniformity, a sample should be taken from each cement shipment and kept in a full, sealed container at least 6 months or until the shipment is exhausted.

5.2.5 Mill certifications—Mill certifications should accompany each shipment that contains the material compositions as well as conformance to the required ASTM standard tests.

5.3—Aggregates for structural or backup concrete

Normalweight or lightweight aggregates conforming to ASTM C33/C33M-11 or C330/C330M-09, respectively, should be used in backup or structural concrete for precast panels.

Aggregates for backup concrete should be stored in clean, well-drained areas in identifiable bins. Bins should be designed to avoid segregation, contamination, or intermixing of different aggregates or aggregate sizes. When aggregate is intended for use with self-consolidating concrete (SCC), gradation, specific gravity, and moisture content are critical and should be measured and monitored frequently.

5.4—Facing aggregates

5.4.1 Grading

5.4.1.1 General—Uniform aggregates for regular concrete are usually selected by standard sieve sizes to provide a balance of fine and coarse sizes. Grading that combines aggregate sizes to produce the maximum weight of aggregate per unit volume of concrete is preferred. Most concrete mixtures are chosen with this in mind but are often limited on the upper end of the coarse aggregate size by dimensions of the panel to be cast, clear distance between reinforcement, clear distance between the reinforcement and form surface, desired finish, and flow requirements.

5.4.1.2 Gap grading of facing aggregates—Because precast concrete panels frequently use exposed aggregate,

Table 5.4.1.3—Typical industry size specifications for exposed aggregate

Percent of indicated size aggregate passing					
Sieve opening		Size D 1-3/8 to 7/8 in. (35 x 22 mm)	Size C 7/8 to 1/2 in. (22 x 13 mm)	Size B 1/2 to 1/4 in. (13 x 6 mm)	Size A 1/4 to 3/32 in. (6 x 2 mm)
in.	mm	Percent of indicated size aggregate passing			
1-1/2	38	100	—	—	—
1-3/8	35	95 to 100	—	—	—
1	25	30 to 60	100	—	—
7/8	22	20 to 40	95 to 100	—	—
5/8	16	0 to 10	30 to 50	100	—
1/2	13	—	10 to 25	95 to 100	—
3/8	9	—	0 to 10	40 to 70	100
1/4	6	—	—	5 to 20	95 to 100
1/8	3	—	—	0 to 10	15 to 35
3/32	2	—	—	—	0 to 10

the desired surface finish, appearance, and texture can dictate fine and coarse aggregate grading. Gap grading can achieve a consistent, uniform panel face with a maximum of aggregate surface exposed. A gap-graded combination of fine or coarse aggregates has one or more sizes missing from the range of standard particle sizes. Producers may elect tighter or more restrictive gradings in an attempt to improve uniformity. Common sizes of gap-graded fine aggregates are 30 to 50 mesh (600 to 300 mm) and 16 to 30 mesh (1.18 mm to 600 mm). Using coarse and fine sizes combined can produce a gap-graded combination that results in less segregation and more uniform surface finish. In SCC, gradation and aggregate proportions may be dictated more by an effort to achieve a stable mixture than the finish. Selecting an aggregate and its properties should satisfy flow requirements and surface finish.

5.4.1.3 Schedule of sizes—Table 5.4.1.3 shows four different size gradings established by precast industry aggregate suppliers for use in exposed aggregate precast concrete. This size schedule is not universally recognized, and some aggregate producers may have their own standards. Panel producers should be aware that small aggregates, 1/8 in. (3 mm) and smaller, may pull out during exposed aggregate surface finishing.

5.4.2 Types and quality of facing aggregates

5.4.2.1 General—Usually decorative facing aggregates are used only in exposed panel faces because of cost. Face layer thickness depends on aggregate size but should be sufficiently thick to prevent backup concrete from showing through the exposed face and eliminate the reinforcing footprint. Face concrete thickness should be 1.5 times the maximum size of coarse facing aggregate but not less than 1 in. (25 mm). Facing aggregate mixtures should be stocked in sufficient quantities from the particular source to complete an entire project. Inappropriate planning may cause unwanted changes in color or texture.