This guide presents recommendations for precast wall panels. This guide should be used with ACI 318 “Building Code Requirements for Reinforced Concrete” which may be legally binding. In addition to a discussion of the basic principles of design, tolerances and materials, this guide also discusses fabrication, installation, quality requirements and testing.

Keywords: admixtures; aggregates; architectural concrete; coatings; colored concrete; concrete finishes; cracking (fracturing); curing; deflection; design; drying shrinkage; erection; exposed aggregate concrete; fabrication; formwork; inspection; joints (junction); precast concrete panels; quality control; repairs; sealants; structural design; sandwich panels; surface defects; temperature; tests; texture; tolerances; volume change; walls.

CONTENTS

Chapter 1-General considerations, pg. 533R-2
  1.1-Introduction
  1.2-Purpose and scope
  1.3-Responsibility for precast concrete wall panels
  1.4-Esthetic considerations

Chapter 2-Wall panel design, pg. 533R-4
  2.1-Introduction
  2.2-Design guidelines
  2.3-Effective dimensions
  2.4-Limiting dimensions for wall panels
  2.5-Serviceability considerations
  2.6-Connections and connection assemblies
  2.7-Provision for architectural features

Chapter 3-Tolerances, pg. 533R-9
  3.1-General
  3.2-Definitions
  3.3-Reasons for tolerances
  3.4-Roles of the engineer-architect
  3.5-Product tolerances for wall panels
  3.6-Erection tolerances for wall panels
  3.7-Interfacing considerations
  3.8-Clearances and tolerances for constructibility

Chapter 4-Materials, pg. 533R-22
  4.1-Introduction
  4.2-Portland cement
  4.3-Aggregates for structural or backup concrete
  4.4-Facing aggregates
  4.5-Admixtures
  4.6-Insulating materials
  4.7-Reinforcement

ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be phrased in mandatory language and incorporated into the Project Documents.
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 itself 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. In addition, by exposing decorative aggregates, using veneer facing materials, and by varying sizes, shapes and textures of panels, the engineer-architect has significant esthetic possibilities for creative response to client needs.

1.2-Purpose and scope

This document provides guidelines for specifying, planning, designing, manufacturing, 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 on a number of projects. Tilt-up concrete, as discussed by ACI 551, is a variation of site precasting. Guidance offered in this document should aid in establishing and maintaining quality site production as well as plant production of precast wall panels.

The guide covers two classes of panels, either both non-load-bearing or load-bearing, fabricated of either normal or lightweight concrete. The panels may be either of the following types:

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

In addition to reinforced panels, lightly prestressed (effective prestress, after all losses, between 150 and 225 psi) and prestressed panels are covered. Structural design considerations briefly addressed in Chapter 2 include the use of panels as shear wall components.

This guide is a compilation of information contained in several earlier ACI Committee 533 reports, a symposium volume, committee member experience and new information and developments in the industry since the committee published its reports.

Heavy emphasis is placed on wall panels with an integral exposed aggregate concrete surface finish. Smooth wall panels, as well as those having finishes of a textured or shaped architectural surface, are included. Panels having natural stone veneer or ceramic veneer finishes are not covered in detail.

1.3-Responsibility for precast concrete wall panels

1.3.1 General — Contractual agreements should assign responsibilities so as to avoid later debate and controversy. This can be particularly troublesome when parties involved disagree 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 has been published. This report makes recommendations on assignment of authority and responsibility for design and construction of precast concrete structures.

This guide covers the design of panels by the design professional, referred to as the engineer-architect*. As defined by ACI 117, engineer-architect or architect-engineer refers to the "architect, the engineer, architectural firm, engineering firm, issuing project drawings and specifications, or administering the work under contract specifications and drawings, or both."

---

* As defined by ACI 117, engineer-architect or architect-engineer refers to the "architect, the engineer, architectural firm, engineering firm, issuing project drawings and specifications, or administering the work under contract specifications and drawings, or both."
throughout the text. Since there are minimum design requirements and methods of design peculiar to precast concrete wall panels, Committee 533 presents supplemental design guidelines which should be used with ACI 318, the provisions of which may be legally binding. Handling and erection procedures vary widely, and guidelines for these operations should correspond with local practices but be consistent with Chapter 2 of this guide. Overlapping responsibilities for the structural design of wall panels may introduce conflicts between engineer-architect and general contractor, regarding shop drawing review, design for handling, erection 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. Since most precasters maintain an engineering staff to prepare shop drawings, the engineer-architect should interact with this group to obtain constructive advice and suggestions concerning local practice, production details, and manufacturing 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 not be possible.

1.3.3 Reinforcement for handling and erection - It is common practice for the engineer-architect to rely on the manufacturer for development of handling techniques and for providing any additional reinforcement required to withstand handling or erection stresses. The engineer-architect may wish to review calculations for handling stresses.

The contract documents may require the manufacturer to accept responsibility for design of panels to resist the loads shown on the engineer of record’s design drawings, provided sufficient information is shown on these drawings, and to resist other loads that occur during stripping, handling, shipping and erection. In this case, it is common for the contract documents to require that the design calculations and erection drawings provided by the panel manufacturer be signed by a professional engineer who is either retained or employed by the manufacturer.

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. Manufacturers, during the preparation of shop drawings, should be given the opportunity to redesign the connections if redesign will achieve more economical details that facilitate manufacture or erection. The manufacturer should review the connections designed by the engineer-architect for structural adequacy and all connection redesign or any other problem noted should be brought to the attention of the engineer-architect. Any deviation from or discrepancy in the approved erection drawings should be noted by the erection contractor prior to the start of erection. The general contractor should make all necessary arrangements for corrections to be made by the parties involved prior to start of erection.

1.3.5 Handling and erection responsibilities - Responsibility for panel erection and cleaning, joint treatment, and supply of hardware needed for handling, attachment, and bracing should be clearly defined in the contract documents. However, contract document specifications, and the specifier, should not prescribe one subcontract because general contractors are generally more knowledgeable of the skills and experience of the various subcontractors who can perform the services, and general contractors can more easily evaluate the economies of the different alternatives.

1.3.5.1 Cleaning - Specifications that require clean panels after installation are recommended. Cleaning need not be the object of a separate operation (see Section 6.5.2). The precast manufacturer and/or carrier are responsible for delivering clean panels. After installation of panels, the responsibility for protecting panels from soiling and staining during subsequent operations should appropriately 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
- provided loose at the job site for connection purposes.

The responsibility for supplying items to be attached to or placed in the structure to receive precast concrete units depends on the type of structure and on local practice. Specifications should indicate who is responsible for the supply and installation of hardware. When the supporting frame is structural steel, erection 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 general 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 must 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 must be called to the engineer-architect’s attention for consideration and cor-
Changes, other than adjustments within the prescribed tolerances, can only be made after approval. Any adjustments affecting structural performance must be approved by the engineer of record. No panel should be left in an unsafe support condition.

1.3.6 Shop drawing approval - Erection and shape drawings prepared by the precast manufacturer (see Section 5.1) should be forwarded to the general contractor for approval as to constructibility and then forwarded to the engineer-architect who checks for conformance with the design requirements and contract documents. Reviewed drawings from the engineer-architect should be returned to the manufacturer with a statement resembling one of the following notations:

1. Approved for conformance with the contract documents. No resubmissions necessary.
2. Approved, as noted, for conformance with the contract documents. No resubmissions necessary.
3. Not approved; revise and resubmit.
4. Rejected.

1.4-Esthetic considerations
The manufacturing techniques and procedures covered in this guide allow flexibility during manufacturing to achieve uniform esthetic results and concrete quality. The use of performance specifications for the appearance of precast wall panels has not been completely successful, due to the difficulty of explaining esthetic requirements or of establishing understandable criteria for acceptance. It is recommended that reference samples be used in determining product characteristics and quality, rather than writing restrictions which may prohibit the manufacturer from using a process that offers the best possibility of producing the desired panel.

1.4.1 Design reference samples - Although full-size sample panels are preferred, some construction specifications may require that the color and texture match small samples. Such samples should be at least 12 x 12 in. although larger samples may be desirable. If both faces of the panel are to be exposed, the 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 of 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 Full-size samples – Committee 533 recommends that at least 3 full-sized sample panels 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-size sample should be kept at the point of manufacture (precasting plant) or at the job site for comparison. Approved full-size panels should be allowed to be used in the completed structure. If full-size samples are required prior to or at the beginning of manufacturing, lead time is necessary and the construction schedule must be adjusted accordingly. When full-size sample panels are not specified, the first production panels should be submitted for inspection and approval by the engineer-architect.

CHAPTER 2-WALL PANEL DESIGN

2.1-Introduction
2.1.1 Scope - This guide presents design recommendations for both prestressed and conventionally reinforced concrete wall panels. Both load-bearing and non-load-bearing panels are covered.

2.1.2 Notation - The standard ACI 318 notation is used throughout this guide. Terms common to ACI 318 but used in this chapter with special application to wall panels are:

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

2.1.3 Definitions – 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 below. Each may be either prestressed or conventionally reinforced.

Panel classes:

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

Load-bearing panel-A precast wall panel that is designed to carry loads from one structural element to other structural elements; load-bearing panels must 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.
Panel types:
- Solid panel-A panel of constant thickness; an allowance for surface texture must be made in determining effective thickness.
- Hollow-core panel-A precast panel that has voids within the thickness in one direction for the full length of the panel.
- Sandwich panel-A precast panel consisting of two layers of concrete separated by a nonstructural insulating core.
- Ribbed panel-A precast panel consisting of a slab reinforced by a system of ribs in one or two directions.

2.2-Design guidelines

2.2.1 General- Precast wall panels should be designed according to Chapters 8, 9, 10, 11, 12, 16, and 18 of ACI 318 except as modified in Sections 2.2.3, 2.2.4.2, 2.2.5, 2.3, 2.4.2, 2.5.2 and 2.5.3 of this recommendation. ACI 318 requirements may be legally binding.

2.2.2 Forces for design - Precast wall panels should be designed to resist all of the following forces wherever applicable:
- Forces developed from differential support settlement, deformations from creep and shrinkage, structural restraint and the effects of environmental temperatures.
- Forces due to construction, handling, storage, transportation, erection, impact, gravity dead and live loads, as well as lateral loads from soil, hydrostatic pressure, wind, and seismic action.
- Local stress concentrations in the vicinity of connections and applied loads must be considered.
- Forces developed from thermal movement or bowing as well as volume change of the panel, with respect to the supporting structure, must be considered.

2.2.3 ACI 318 provisions applicable for member design - The following sections of ACI 318 should be followed for the design aspects enumerated, except as otherwise modified in this guide:

- Effective prestress-ACI 318, Section 18.6. The average concrete stress due to prestressing after losses is limited to a range of 150 to 800 psi.
- Flexure-ACI 318, Chapter 10 for nonprestressed panels and ACI 318, Chapter 18 for prestressed panels. Requirements of ACI 318, Section 10.7 for deep beams apply regardless of whether the member is prestressed or nonprestressed.
- Shear-ACI 318, Chapter 11 for both prestressed and nonprestressed panels.
- Bearing-ACI 318, Sections 10.15 and 15.8.
- Combined bending and axial load-ACI 318, Sections 10.3 and 10.115.

2.2.4 Combined bending and axial load

2.2.4.1 General - All forces listed in Section 2.2.2 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 a rational analysis of the structure. Considerations of member and/or joint translation should be considered in the analysis.

In lieu of the procedure described above, compression member design may be based on the approximate procedures given in Section 2.2.4.2.

2.2.4.2 Approximate evaluation of slenderness effect — Procedures described in ACI 318, Section 10.11 should be followed for determining the unsupported length, effective length, and radius of gyration of precast wall panels.

a) The effects of slenderness may be neglected if the slenderness conforms to ACI 318, Section 10.11.4.1 or 10.11.4.2. For compression members with slenderness $kЕ_{fl}/r$ greater than 150, an analysis according to Section 2.2.4.1 of this guide should be made.

b) The magnified moment for design of a compression member should be determined according to ACI 318, Section 10.11.5.1.

c) For precast wall panels considered to be reinforced concrete compression members by these recommendations, the provisions of ACI 318, Section 10.11.5.2 can be used in lieu of more accurate calculations.

d) For precast wall panels considered to be prestressed concrete compression members by these recommendations, the provisions of Section 3.5 of the PCI Design Handbook, can be used in lieu of more accurate calculations.

e) An equivalent uniform bending moment factor, defined in accordance with ACI 318, Section 10.11.5.3 should be considered for precast wall panels braced against sideways and without transverse load between supports.

f) The minimum eccentricity, according to ACI 318, Sections 10.3.5, 10.3.6, 10.11.5.4 or 10.11.5.5, as appropriate, should be considered for precast wall panels when no bending moment occurs at either end of the panel.

2.2.5 Reinf orce ment- Precast wall panels are not required to have lateral hoop or spiral reinforcement unless analysis or experience indicates this reinforcement is required.

Limits of reinforcement for precast wall panels should conform to ACI 318, Sections 7.10, 7.12, 10.9, 14.3, and 18.11, except that the minimum ratio of reinforcement area to gross concrete area should not be less than 0.001. Two-way reinforcement is not required for some essentially one-way panels, such as hollow-core panels.

2.3-Effective dimensions

2.3.1 Effective thickness

2.3.1.1 General- The effective panel thickness for
Exposed aggregate surface
Architectural facing concrete
Bonded interface
Structural backup concrete

Depth of reveal
Total panel thickness (nominal)

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

or

\( h_{\text{eff}} = \text{Total panel thickness} \)

Fig. 2.3.1.2-Effective thickness of architectural faced panels

\( I_g = \text{Uncracked moment of inertia} \)

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

Fig. 2.3.1.3-Effective thickness of solid, hollow-core, or ribbed panels

design may be different from the total panel thickness. The following sections explain how to determine the effective thickness for design purposes and Figs. 2.3.1.2, 2.3.1.3 and 2.3.1.4 provide the general characteristics of the various effective thicknesses.

2.3.1.2 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\% of the total thickness. The effective thickness of a wall panel with a noncomposite facing should not include the separate facing thickness.

2.3.1.3 Solid, hollow-core, and ribbed panels - The effective panel thickness should be determined by Eq. (2-1).

\[ h_{\text{eff}} = \frac{3 \sqrt[3]{\frac{12 \, I_g}{b}}}{\sqrt[3]{b}} \] (2-1)

where \( I_g \) is the uncracked moment of inertia accounting for voids or ribs, if they exist.
Fig. 2.3.1.4-Effective thickness of sandwich panels

2.3.1.4 Sandwich panels - The 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. (2-1).

If the insulation core is cellular lightweight concrete or lightweight concrete made with mineral aggregates, the shear transfer through the insulation core must 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. See Section 2.4.2 for limitations on the maximum slenderness ratio of the load-bearing wythe.

2.3.1.5 Panels of irregular shape - Panels not conforming to the configurations listed in this section may have the effective thickness determined by analysis or testing.

2.3.2 Effective width - If concentrated loads or bending moments are applied to the top and bottom of a wall panel, the effect of local stress in the vicinity of the applied concentrated load or bending moment should be investigated. The effective width should be determined by a rational analysis.

In lieu of a rational analysis, the effective width for a concentrated load may not exceed the center-to-center distance between supports, nor the width of the loaded portion plus six times the wall panel effective thickness on each side of the concentrated load.

In lieu of a rational analysis, the effective width for concentrated bending moments may not exceed the effective thickness of the wall panel or the width of the corbel 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.

2.4-Limiting dimensions for wall panels
2.4.1 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.

2.4.2 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 \((kC/d)\) of a precast wall panel should not exceed 200.

The 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.

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.

2.5-Serviceability considerations
2.5.1 General — The action of service loads on deflections perpendicular and parallel to the wall panel must be considered. Fatigue, impact (if any), cracking, and in-plane lateral stability at service load conditions must be accounted for in design.

2.5.2 Computed permissible deflections — Precast wall panel dimensions should be chosen so that under service load conditions, the deflection of any point on the panel measured from its original position should not exceed the limits given in Table 2.5.2. In calculating the deflection, the nonlinear behavior of the materials and/or the structural member should be recognized.

Table 2.5.2-Deflection limits for precast wall panels

<table>
<thead>
<tr>
<th>Type of member</th>
<th>Deflection to be considered</th>
<th>Deflection limitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load-bearing precast wall panels</td>
<td>Immediate deflection due to combined effects of prestress, if any, self weight, and superimposed dead load. Immediate deflection due to live load, in.</td>
<td>1/240 but not greater than (\frac{3}{4}) in. 1/360 but not greater than (\frac{3}{4}) in.</td>
</tr>
<tr>
<td>Non-load-bearing precast wall panel elements likely to be damaged by large deflection</td>
<td>That part of the total deflection after the installation of the non-load-bearing element (the sum of the long time deflection due to all sustained loads and the immediate deflection due to live load)</td>
<td>1/480 but not greater than (\frac{3}{4}) in.</td>
</tr>
</tbody>
</table>

2.5.3 Cracking
2.5.3.1 Acceptability of cracking - Although precast wall panels typically undergo far less cracking than cast-in-place concrete, 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 must be evaluated. If the crack width is narrow, not over 0.010 in., the structural adequacy of the casting will remain unimpaired, as long as corrosion of the reinforcement is prevented. Therefore, if the reinforcement is coated for corrosion resistance, wall panels containing cracks up to 0.005 in. wide for surfaces exposed to weather and 0.010 in. wide for surfaces not exposed to the weather should be acceptable. The limitation on crack size specified is for structural reasons. The esthetic limitation will depend on the texture of the surface and the appearance required. 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. For smooth surfaces of high quality it may be desirable to limit cracking in interior panels to 0.005 in. In addition, it should be noted that cracks will become even more pronounced on surfaces receiving a sandblasted or acid etch finish.

Additional guidance on cracking and its causes can be found in the PCI Quality Control Manual, PCI Design Handbook, PCI Architectural Precast Concrete, ACI 224.1R, and ACI 224R.

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 0.01 in. 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 surface of concrete. These probably occur in many panels, but they are not readily visible in exposed aggregate surfaces, or when the concrete is dark. They are more apparent on white panels, flat surfaces, and smooth finishes. Crazing cracks are of little structural importance and should not be cause for rejection. If the panels are to be installed in an environment that may be the source of considerable soil- ing, it may be advisable to avoid smooth concrete finishes in order to render the potential crazing less visible.

2.5.3.2 Crack prevention and control - Significant reductions in crack widths can be obtained by properly selecting and locating reinforcement and by maintaining accurate positioning of the steel during the casting operation. Reinforcement is more effective if it consists of more closely-spaced, smaller diameter bars or wire, particularly in thin sections. For this reason, welded wire fabric reinforcement is commonly used instead of reinforcing bars because of the relatively close spacing, 4 to 6 in. or less, of the wires.

The flexural reinforcement distribution requirements in ACI 318, Section 10.6 should be followed for reinforced precast or architectural wall panel surfaces not exposed to view. If the geometry of the precast member is more like that of a two-way slab, flexural reinforcement requirement of ACI 318 Section 10.6 may lead to crack widths wider than expected.

2.5.3.3 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 \(5\sqrt{f_{c}}\). The value of the tensile strength of concrete should be modified according to ACI 318, Section 11.2 if lightweight aggregate concrete is used.

2.6-Connections and connection assemblies

2.6.1 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 code.

Whenever possible, panels should be concentrically supported to avoid bowing and warping of panels 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 must be given to the additional loads required for the design of the connection or connections.

Concepts for design of connections for precast wall panels may be found in the PCI Design Handbook and the PCI Design and Typical Details of Connections for Precast and Prestressed Concrete.

2.6.2 Panel movement - Wall panel connection assemblies should be designed to allow for panel movement caused by volumetric change in the concrete, induced by temperature, moisture differential, and creep in prestressed panels, as well as by differential movement or drift between the building frame and wall panel units. Guidance on the design for these conditions can be found in PCI Design Handbook and Ref. 7.

2.6.3 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.

The designer should avoid hanging the 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 their proper installation.
2.6.4 Haunches – Concrete haunches used to positively seat panels should conform to shear requirements of ACI 318, Section 11.9 and should be designed for eccentric loading and combined shear, bending, tension, and bearing stresses. The effect of eccentricity which will cause the panel to deflect should be considered in the design of panel reinforcement.

2.6.5 Panel inserts – The design of wall panel inserts that are part of a connection assembly should be based on design relationships incorporating the load factors and strength reduction factors (φ factors) specified in ACI 318. The connections should not be the weak link in a precast system. Inserts should have a factor of safety consistent with the insert manufacturer’s recommendation.

2.6.6 Fire resistance – Wall panel connections should be fireproofed as required by local codes and have minimum fire resistance equivalent to that required by code for the wall panels.

2.6.7 Weld design – Potential relative movement between the panel and supporting structural frame or adjacent panels should be investigated when designing the 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.

2.7-Provision for architectural features

2.7.1 Glass staining or etching – Glass, like all building materials, is subject to the effects of weathering. When a moist material is in contact with or applied to glass, the glass surface may undergo subtle changes in the contact area. If the coating in contact with the glass is inert and moistureproof, the glass surface will be protected from changes caused by exposure to moisture. However, if the coating material is removed, a differential surface change may become quite visible and unattractive under some lighting and viewing conditions, even though the change is slight. Finely divided damp materials, for example, dirt and dust, in contact with glass can cause the glass constituents to dissolve slightly and be redeposited at an evaporating edge resulting in staining. In addition, some silicone sealants have ingredients that may leach out and stain the glass.

When glass (sodium calcium silicate) is exposed to moisture, a minute amount of the glass will dissolve. If the dissolved material is washed away, little change can be seen by the human eye. But when the solution remains on the glass, atmospheric carbonation of the alkali and alkaline earth silicates causes a subsequent deposit of silica gel. The gel on aging and exposure to atmospheric acids becomes difficult to remove. When this happens uniformly, the eye does not detect the differences. However, the silica gel deposit, or the glass etch depth need not be thicker than a wavelength of light for the eye to detect it. Frequent washing of the windows tends to remove the gel before it becomes hard, minimizing staining and etching.

Directed slow-water runoff and the resultant dirt accumulation cause the glass to be attacked nonuniformly, and eventually the cycle of water drying, gel forming, acid atmosphere attack, and alkali washing compounds, causes in-depth glass dissolution; no amount of cleaning or buffing will remove the stain or etch.

Staining will be more noticeable on tinted heat-absorbing glass because of the greater contract between the light color of the stain or etch and the darker color of the glass. There is no known difference in the composition of tinted glasses, which contributes to this staining, as compared to clear glass.

2.7.2 Drip details – Directed slow-water runoff of rainwater over building facades and dirt accumulation sometimes contributes to staining or etching of glass surfaces. This phenomenon was briefly discussed in Section 1.4 and is more fully explained in PCI Architectural Precast Concrete. Appropriate building details can reduce the amount of 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 washdown of the glass should be anticipated.

The drip section should be designed in relation to the slope of the concrete surface (see Fig. 2.7.2.1). 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. The introduction of edge drips and a second drip or gutter serve 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 the use of extrusions (either aluminum or neoprene) across the head of the window, which have either an integral gutter or an extended drip lip of at least 1 in. also shown on Fig. 2.7.2.1.

2.7.3 Joint size and location – Joints between precast panels or panels and adjacent building materials must be wide enough to accommodate anticipated panel and building movements. No joint should ever be designed to be less than \(\frac{3}{8}\times\frac{3}{8}\) in. Particular care must be given to joint tolerances in order for the joint sealant system to perform within its design capacities. For optimum performance and maximum sealant life, recommendations of the sealant manufacturer should be followed.

Panels less than 15 ft long may have 1/2-in. joints, but all other panels should have at least 3/4-in. joints. Corner joints should be 1/4 in. wider to accommodate the extra movement and bowing that occurs there. Joint widths of 3/8 in. are considered highly risky for any sealant installation. When joints are too narrow, adjacent panels or building materials may come in contact and be subject to induced loading, distortion, cracking, and crushing of ends.

CHAPTER 3-TOLERANCES

3.1-General

Precast structures should be designed and detailed in
such a manner that the complete structure will be safe, functional, aesthetically appealing, and economical. However no structure is exactly level, plumb, straight and true. All construction and materials should be specified with permissible variations, or tolerances, limiting the extent of deviation from design values. These tolerances require monitoring in order to construct the structure as designed. General construction tolerances for cast-in-place and precast concrete have been summarized by ACI 117 and the PCI Committee on Tolerances. This chapter presents tolerances that are 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 3.5)
- Erection tolerances (Section 3.6)
- Interfacing tolerances (Section 3.7)

When tolerances are understood and provided for in the design stage, the task of determining and specifying them is made easier. The precaster, constructor, and erector must all understand the type of allowances made in the design stage in order to construct the structure as designed.

3.2-Definitions

**Bowing**—An overall out-of-plane distortion, differing from warping, in that while two edges of the panel may fall in the same plane, the portion of the panel between the edges can be out of the plane defined by the edges. Several bowing conditions are shown in Fig. 3.2.1.

**Differential bowing** may be observable when panels are viewed together on the completed structure. When two panels bow in the same direction, the magnitude of differential 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 differential bowing is the algebraic difference.

For example in Fig. 3.2.2 if the maximum bowing of panel 3 was +1/4 in. and the maximum bowing of panel 4 was −1/4 in., then the differential bowing between these two adjacent panels is 1/2 in.

**Camber**—The maximum deviation in elevation from a straight line through the end points of an element; a camber deflection that is intentionally built into a structural element or formed to improve appearance or to nullify the deflection of the element under the effects of loads, shrinkage, and creep.

**Clearance**—Interface space between two members is called clearance. Clearance is normally specified to allow for the differing amounts of deviation that can occur within a tolerance envelope and to allow for anticipated
movement caused by volume change, temperature effects, or elastic deflection.

Dimensions-There are basic (nominal) and actual dimensions. The basic dimension is shown on the contract drawings or called for in the specifications. These dimensions apply to size, location, and relative location of the precast member within the structure. The actual dimension is the measured dimension after casting or installation of the precast member.

Level-A line or plane perpendicular to plumb.

Plumb-A vertical direction radiating from the center of the earth, commonly determined by a suspended weight.

Skew-An out-of-square variation from a rectangular shape. This is normally measured by comparing the length of the diagonals.

Surface out-of-planeness-A local smoothness variation rather than a bowing variation. The tolerance for this variation is usually expressed in fractions of an inch or in inches per 10 ft. The tolerance is usually checked with a 10-ft straightedge or equivalent as shown in Fig. 3.2.3.

Tolerance-A permitted variation from the basic dimension or quantity as in the length, width, or depth of a member; the range of variation permitted in maintaining a basic dimension should be specified.

Variant-The difference between the actual dimension or location and the basic dimension. When the permitted variation is symmetrical, the tolerance can be expressed as a plus-minus (±) variation from a specified dimension or relationship.

Warping-A deviation of the panel from its original shape or the overall variation from planeness in which the panel corners do not fall within the same plane. Warping tolerances are stated in terms of the magnitude of the corner variation as shown in Fig. 3.2.4. This value is usually given in terms of the allowable variation per foot of distance from the nearest adjacent corner with a “not-to-exceed” maximum value of corner warping.

3.3-Reasons for tolerances

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

Structural Considerations-To ensure that the structural design properly accounts for factors sensitive to variations in dimensional control. Examples include eccentric loading, bearing areas, hardware and hardwar anchorages, locating, and locating of reinforcing or prestressing...
steel.

Performance-To ensure acceptable performance of joints and interfacing materials in the finished structure.

Appearance-To ensure that the deviation from theoretical requirements will be controllable and result in an acceptable appearance. Large deviations are objectionable, whether they occur suddenly or cumulatively.

Cost-To ensure ease and speed of production and erection 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 also to establish responsibility for developing, achieving and maintaining mutually agreed-on tolerance values.

3.4-Role of the 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 manufacturing and erection costs are directly related to degree of precision required. Thus it is economically desirable and practically safer to design with maximum flexibility and to keep tolerance requirements as liberal as possible. Tolerances given 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 particular 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 production errors can be absorbed. The consequences of all tolerances 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. The engineer-architect should in this case specify the minimum bearing dimensions.

Careful inspection of the listed tolerances reveals that many times one tolerance will override another. The permitted variation for one element of the structure should not be such that it would require another element of the structure to exceed its tolerances. Restrictively small tolerances should be reviewed by the precaster and general contractor to ascertain that they are compatible with other elements and that they can in fact be met. For example, a requirement which states that “no bowing, warping or movement is permitted” is not practical. All involved in the design-construction process should understand that tolerances given 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 the structural integrity or architectural performance of the member.

b) The member can be brought within tolerance by structurally and architecturally satisfactory means.

c) The total erected assembly can be modified to meet all structural and architectural requirements.

3.5-Product tolerances for wall panels

3.5.1 General – Product tolerances cover the dimensions and dimensional relationships of individual precast concrete members. All tolerances should be based on a degree of accuracy which is practical and 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.

Manufacturing tolerances are standardized throughout the precast industry and for economic reasons should be made more exacting 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 percent of typical tolerances to avoid joint shadows. When design details lead to an alignment problem or provide inadequate joint size, the product tolerance may have to be adjusted to compensate for the 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, the dimensions of openings between panels, and perhaps the overall length of the structure.

- Panels out of square can cause tapered joints and make adjustment of adjacent panels extremely difficult. Sealant application difficulties due to tapered joints can lead to future 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.

3.5.2 Dimensional tolerances – Architectural precast concrete panels should be manufactured and installed so that the face of each panel which is exposed to view after erection complies with the dimensional requirements shown in Fig. 3.5.2a. Figure 3.5.2a also shows 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 relatively close to their
a = Overall height and width measured at the face adjacent to the mold at time of casting or neutral axis of ribbed members:
- 10 ft or under
- 10 ft to 20 ft
- 20 ft to 40 ft
- Each additional 10 ft

b = Thickness, total or flange

c = Rib width (thickness)
d = Rib to edge of flange
e = Distance between ribs

f = Variation (deviation) of plane of side mold

$\pm 1/8$ in. for 3 in. of depth or $\pm 1/16$ in. total, whichever is greater

$\pm 1/8$ in. per 6 ft of diagonal or $\pm 1/2$ in. total, whichever is greater

$\pm 1/4$ in.

$\pm 3/4$ in.

$\pm 1/4$ in.

$\pm 1/8$ in.

$L/360$, maximum 1 in.

$1/2$ in.

$1/4$ in. in 10 ft

o = Warping

1/16 in. per ft of distance from nearest adjacent corner, maximum 1 in.

p = Location of window opening

$\pm 1/4$ in.

q = Position of plates

$\pm 1$ in.

r = Tipping and flushness of plates

$\pm 1/4$ in.

POSITION TOLERANCES: For cast-in items, measured from the datum line location as shown on the approved erection drawings

Inserts

$\pm 1/2$ in.

Weld plates

$\pm 1$ in.

Handling devices

$\pm 3$ in.

Reinforcing steel and welded wire fabric**

$\pm 1/4$ in.

Tendons

$\pm 1/8$ in.

Flashings reglets

$\pm 1/4$ in.

Flashings reglets, at edge of panel

$\pm 1/8$ in.

Reglets for glazing gaskets

$\pm 1/8$ in.

Groove width for glazing gaskets

$\pm 1/16$ in.

Electrical outlets, hose bibs, or other utility embedded items

$\pm 1/2$ in.

Openings and blockouts

$\pm 1/4$ in.

Center line of blockout*

$\pm 1/4$ in.

Haunches, to be placed in form

$\pm 1/4$ in.

*Some types of window and equipment frames require openings more accurately placed, and when this is the case, the minimum practical tolerance should be defined with the input of the producer.

**Tolerance given should be used where position has structural implications or affects concrete cover, otherwise use $\pm 1/2$ in.

Fig. 3.5.2a-Production tolerances for precast architectural wall panels
CROSS SECTION

\begin{align*}
a &= \text{Length} & \pm \frac{1}{2} \text{ in.} \\
b &= \text{Width} & \pm \frac{1}{4} \text{ in.} \\
c &= \text{Depth} & \pm \frac{1}{8} \text{ in.} \\
d &= \text{Stem width} & \pm \frac{1}{8} \text{ in.} \\
e &= \text{Flange thickness} & \pm \frac{1}{8} \text{ in.} \\
f &= \text{Distance between stems} & \pm \frac{1}{8} \text{ in.} \\
g &= \text{Stem to edge of top flange} & \pm \frac{1}{8} \text{ in.} \\
h &= \text{Variation from specified} & \pm \frac{1}{8} \text{ in. per 12 in. of width, } \pm \frac{1}{4} \text{ in. maximum} \\
i &= \text{Variation from specified} & \pm \frac{1}{8} \text{ in. per 12 in.} \\
j &= \text{Sweep (variation from straight line parallel to center line of member)} & \pm \frac{1}{4} \text{ in.} \\
k &= \text{Position of tendons} & \pm \frac{3}{8} \text{ in.} \\
l &= \text{Position of blockouts} & \pm \frac{1}{4} \text{ in.} \\
m &= \text{Size of blockouts} & \pm \frac{1}{4} \text{ in.} \\
n &= \text{Position of plates} & \pm \frac{1}{4} \text{ in.} \\
o &= \text{Tipping and flushness of plates} & \pm \frac{1}{4} \text{ in.} \\
p &= \text{Position of inserts for structural connections} & \pm \frac{1}{2} \text{ in.} \\
q &= \text{Position of handling devices} & \pm 6 \text{ in.} \\
r &= \text{Bowing} & \frac{L}{360} \text{ maximum*} \\
s &= \text{Differential bowing between adjacent panels of the same design} & \pm \frac{1}{2} \text{ in. (13 mm)*} \\
t &= \text{Position of flashing reglets} & \pm \frac{1}{4} \text{ in.} \\
u &= \text{Haunches (noncumulative)} & \pm \frac{1}{4} \text{ in.} \\
u_1 &= \text{Bearing elevation from bottom of panel} & \pm \frac{1}{4} \text{ in.} \\
u_2 &= \text{Relative position of bearing elevation in vertical plane} & \pm \frac{1}{8} \text{ in.} \\
u_3 &= \text{Haunch bearing surface squareness perpendicular to applied major load} & \pm \frac{1}{8} \text{ in. per 18 in., } \pm \frac{1}{4} \text{ in. maximum} \\
v &= \text{Local smoothness any surface} & \pm \frac{1}{4} \text{ in. in 10 ft*} \\
w &= \text{Warping} & \pm \frac{1}{16} \text{ in. per ft of distance from nearest adjacent corner} \\

*Does not apply to visually concealed surfaces. Refer to Fig. 3.2.1, 3.22 and 3.2.3 for definition.

Fig. 3.5.2b-Dimension (production) tolerances for standard precast ribbed panels used as wall panels
Fig. 3.5.2c-Dimensional tolerances for hollow core slabs used as wall panels
correct location. Misalignment of these reglets at corners, or casting them in a warped or “racked” position will restrict proper installation of the glazing gasket. In addition, gasket manufacturers place very restrictive tolerances on the groove width and surface smoothness necessary to obtain a proper moisture seal of the gasket.

Dimension tolerances for standard precast ribbed panels are shown in Fig. 3.5.2b. Dimension tolerances for hollow-core slabs used as wall panels are shown in Fig. 3.5.2c. Standardized ribbed and hollow-core members, typically used for roof and floor units, are frequently adapted for use as wall panels. The tolerances for these standardized units are generally more liberal than those for architectural panels. If the engineer-architect cannot accept the standard tolerances of ribbed and hollow-core units when using them as wall panels, they should specify other tolerances as required or the architectural tolerances as given in Fig. 3.5.2c.

3.5.3 Warping and bowing - Warping and bowing are defined in Section 3.2. 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 3.2) is differentiated from bowing because it is not a characteristic of the entire panel shape, but rather a local smoothness variation. The tolerance for local smoothness is checked with a straightedge or the equivalent as shown in Fig. 3.2.3. The measurement of warping is shown in Fig. 3.2.4.

Table 3.5.3 shows a relationship between overall flat panel dimensions and cross-sectional thickness. If the thickness is less than suggested in Table 3.5.3, warping tolerances should be reviewed for the possibility of increasing the tolerance. Panels thinner than those in Table 3.5.3 should not automatically be subjected to the standard tolerances for bowing and warping. Ribbed panels and panels manufactured using aggregates larger than 3/4 in. exposed at the surface also need further consideration in the establishment of bowing and warping tolerances. Tolerances for flat panels of nonhomogeneous materials, such as two widely different concrete mixes or natural stone veneer with concrete backup, should be reviewed; these tolerances may have to be increased or reduced to meet design criteria.

3.6-Erection tolerances for wall panels

3.6.1 Discussion-Erection tolerances are required for the functional matching of the precast elements with the building structure. The engineer-architect should set them to be compatible with the desired architectural expression and detail appropriate to the building structure and site conditions. Erection 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. Erection tolerances affect the work of several trades and must be consistent with the tolerances specified for those trades. It is the responsibility of the engineer-architect to see that tolerances are compatible.

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

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

Location or erection tolerances for wall panels should be noncumulative. The recommended tolerances are listed in Figs. 3.6.1 and 3.6.2. Figure 3.6.1 shows erection tolerances for precast wall panels while Fig. 3.6.2 shows erection tolerances for structural wall panels.

3.6.2 Control points and benchmarks - To ensure accurate application of erection tolerances, the general contractor should establish and maintain accurate control points and bench marks, in areas that will remain undisturbed until final completion and acceptance of a project. The contractor should provide the erector with a building perimeter offset line at each floor approximately 2 ft from the edge of the floor slab and bench marks on all perimeter columns. Offset lines and bench marks should

<table>
<thead>
<tr>
<th>Panel dimensions</th>
<th>8 ft</th>
<th>10 ft</th>
<th>12 ft</th>
<th>16 ft</th>
<th>20 ft</th>
<th>24 ft</th>
<th>28 ft</th>
<th>32 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 ft</td>
<td>3 in.</td>
<td>4 in.</td>
<td>4 in.</td>
<td>5 in.</td>
<td>5 in.</td>
<td>6 in.</td>
<td>6 in.</td>
<td>7 in.</td>
</tr>
<tr>
<td>6 ft</td>
<td>3 in.</td>
<td>4 in.</td>
<td>4 in.</td>
<td>5 in.</td>
<td>6 in.</td>
<td>6 in.</td>
<td>7 in.</td>
<td>7 in.</td>
</tr>
<tr>
<td>8 ft</td>
<td>4 in.</td>
<td>5 in.</td>
<td>5 in.</td>
<td>6 in.</td>
<td>6 in.</td>
<td>7 in.</td>
<td>7 in.</td>
<td>8 in.</td>
</tr>
<tr>
<td>10 ft</td>
<td>5 in.</td>
<td>5 in.</td>
<td>6 in.</td>
<td>6 in.</td>
<td>7 in.</td>
<td>7 in.</td>
<td>8 in.</td>
<td>8 in.</td>
</tr>
</tbody>
</table>
Fig. 3.6.1-Erection tolerances for precast wall panels

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>Plan location from building grid datum *</td>
<td>± 1/2 in.</td>
</tr>
<tr>
<td>a1</td>
<td>Plan location from center line of steel **</td>
<td>± 1/2 in.</td>
</tr>
<tr>
<td>b</td>
<td>Top elevation measured from nominal top elevation</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Exposed individual panel</td>
<td>± 1/4 in.</td>
</tr>
<tr>
<td></td>
<td>Nonexposed individual panel</td>
<td>± 1/2 in.</td>
</tr>
<tr>
<td></td>
<td>Exposed relative to adjacent panel</td>
<td>1/4 in.</td>
</tr>
<tr>
<td></td>
<td>Nonexposed relative to adjacent panel</td>
<td>1/2 in.</td>
</tr>
<tr>
<td>c</td>
<td>Support elevation from nominal elevation</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Maximum low</td>
<td>1/2 in.</td>
</tr>
<tr>
<td></td>
<td>Maximum high</td>
<td>1/4 in.</td>
</tr>
<tr>
<td>d</td>
<td>Maximum plumb variation over height of structure or 100 ft, whichever is less*</td>
<td>1 in.</td>
</tr>
<tr>
<td>e</td>
<td>Plumb in any 10 ft of element height</td>
<td>1/4 in.</td>
</tr>
<tr>
<td>f</td>
<td>Maximum jog in alignment of matching edges</td>
<td>1/4 in.</td>
</tr>
<tr>
<td>g</td>
<td>Joint width (governs over joint taper)</td>
<td>± 1/4 in.</td>
</tr>
<tr>
<td>h</td>
<td>Joint taper maximum</td>
<td>3/8 in.</td>
</tr>
<tr>
<td>h10</td>
<td>Joint taper over 10 ft</td>
<td>1/4 in.</td>
</tr>
<tr>
<td>i</td>
<td>Maximum jog in alignment of matching faces</td>
<td>1/4 in.</td>
</tr>
<tr>
<td>j</td>
<td>Differential bowing or camber as erected between adjacent members of the same design</td>
<td>1/4 in.</td>
</tr>
</tbody>
</table>

* For precast buildings in excess of 100 ft tall, tolerances a and d can increase at the rate of 1/8 in. per story over 100 ft to a maximum of 2 in.

**For precast elements erected on a steel frame, this tolerance takes precedence over tolerance on dimension a.
533R-18  ACI COMMITTEE REPORT

PRECAST CONCRETE PANEL

CAST-IN-PLACE OR PRECAST CONCRETE

PLAN

CAST-IN-PLACE CONCRETE

ELEVATION

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>Plan location from building grid datum*</td>
</tr>
<tr>
<td>a₁</td>
<td>Plan location from center-line of steel†</td>
</tr>
<tr>
<td>b</td>
<td>Top elevation from nominal top elevation</td>
</tr>
<tr>
<td>c</td>
<td>Bearing elevation from nominal elevation</td>
</tr>
<tr>
<td>d</td>
<td>Maximum plumb variation over height of structure or 100 ft, whichever is less*</td>
</tr>
<tr>
<td>e</td>
<td>Plumb in any 10 ft of element height</td>
</tr>
<tr>
<td>f</td>
<td>Maximum jog in alignment of matching edges</td>
</tr>
<tr>
<td>g</td>
<td>Joint width (governs over joint taper)</td>
</tr>
<tr>
<td>h</td>
<td>Joint taper over length of panel</td>
</tr>
<tr>
<td>h₁₀</td>
<td>Joint taper over 10 ft length</td>
</tr>
<tr>
<td>i</td>
<td>Maximum jog in alignment of matching faces</td>
</tr>
<tr>
<td>j</td>
<td>Differential bowing, as erected, between adjacent members of the same design†</td>
</tr>
</tbody>
</table>

*For precast buildings in excess of 100 ft tall, tolerances "a" and "d" can increase at the rate of 1/8 in. per story over 100 ft to a maximum of 2 in.

‡For precast elements erected on a steel frame, this tolerance takes precedence over tolerance on dimension “a.”

Fig. 3.6.2-Structural wall panels
be maintained until final completion and acceptance of the work. They may be scored into columns and floor slabs, or laid out as chalk lines and lacquered for protection.

3.6.3 Joint problems — Width variations between adjacent joints can be minimized by setting out joint center lines equally spaced along an elevation and centering panels between them. The larger the panels, the wider the theoretical joint should be in order to accommodate realistic tolerances in straightness of panel edge, in slope of edge, and in 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 possible 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. The 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. joint may be specified with a tolerance of ± 1/4 in. but with only a 3/16 in. 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 ± 3/4 in. 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 have been designed into the construction details and are adhered to, the erection should be able to:

- minimize joint irregularities such as tapered joints (panel edges not parallel)
- minimize jogs at intersections
- minimize nonuniformity of joint width
- maintain the proper opening dimensions
- properly construct all precast connections
- align the vertical faces of the units to avoid offsets
- prevent the accumulation of tolerances.

A more precise installation and general improvement in appearance are thus achieved.

3.7-Interfacing considerations

3.7.1 General — Interface tolerances and clearances are those required for joining of different materials and to accommodate the relative movements between such materials during the life of the building. They cover products installed after the precast members are in place as well as materials installed before precast erection. The engineer-architect should provide for proper clearances (purposely provided space between adjacent independent materials) between the theoretical face of the structure and the back face of the precast element. The face of 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 the precast panels are in place. The clearance space provides a buffer where erection, product, and interface tolerances can be absorbed.

Where matching of the manufactured materials depends on work at the construction site, interface tolerances should equal erection tolerances. Where the execution is independent of site work, tolerances should closely match the standard tolerances for the materials to be joined. Fabrication and erection tolerances of other materials must be considered in design. Precast elements must 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.

3.7.2 Building frame tolerances — Erection tolerances for precast panels are of necessity largely determined by the actual alignment and dimensional accuracy of the building foundation and frame. The general contractor is responsible for the plumbness, level, and alignment of the foundation and structural frame including the location of all bearing surfaces and anchorages for precast products. Many engineer-architects fail to recognize the critical importance of controlling foundation and building frame tolerances. It is not uncommon to find specifications which make no mention of tolerances for the structure to which the precast is connected. Likewise, it is not uncommon to find architectural or structural drawings on which clearance dimensions fail to take into account the normal product and erection tolerances. If precast elements are to be installed plumb, square, and true, the actual location of surfaces affecting the precast elements’ alignment (including the levels of floor slabs and beams, the vertical alignment of floor slab edges and the plumbness of columns or wall) must be known before erection begins.

Concrete cast-in-place frames to which precast elements are attached should meet the tolerances shown in Table 3.7.2 in addition to ACI 301 or ACI 117 requirements. Greater variations in height of floors are more prevalent in cast-in-place structures than in other structural frames. This affects location or mating of the insert in the precast with the cast-in-place connection device. Tolerances for cast-in-place structures may have to be increased further to reflect local trade practices, the complexity of the structure, and climatic conditions.

Figure 3.7 shows erection tolerances for beams and spandrels, particularly precast element to precast element, precast to cast-in-place concrete and masonry, and precast to steel frame.

3.7.3 Mixed construction — It should be recognized that ACI 117 applies only to reinforced concrete and ma-
Table 3.7.2—Supplementary tolerances for cast-in-place concrete frames to which precast concrete is to be attached

<table>
<thead>
<tr>
<th>Category</th>
<th>Tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Footings, caisson caps, and pile caps</td>
<td>± ½ in.</td>
</tr>
<tr>
<td>Variation of bearing surface from specified elevation</td>
<td></td>
</tr>
<tr>
<td>Piers, columns, and walls</td>
<td></td>
</tr>
<tr>
<td>Deviation from the level or grades specified in the drawings</td>
<td>± ½ in.</td>
</tr>
<tr>
<td>Any bay or wall length less than 20 ft</td>
<td>± ¼ in.</td>
</tr>
<tr>
<td>Any bay or wall length greater than 20 ft</td>
<td></td>
</tr>
<tr>
<td>Deviation from column cross-sectional dimensions or wall thickness</td>
<td>+ ½ in., - ¼ in.</td>
</tr>
<tr>
<td>Anchor bolts</td>
<td>± ¼ in.</td>
</tr>
<tr>
<td>Variation from specified location in plan</td>
<td></td>
</tr>
<tr>
<td>Anchor bolt projection</td>
<td>+ ½ in., - ¼ in.</td>
</tr>
<tr>
<td>Plumbness of anchor bolt</td>
<td>± ⅛ in. per ft</td>
</tr>
<tr>
<td>Floor Elevations</td>
<td>± ¼ in. in 10 ft</td>
</tr>
<tr>
<td>Variation from specified level</td>
<td>±½ in. in 30 ft or greater length</td>
</tr>
</tbody>
</table>

sonry buildings, and the AISC Code of Standard Practice only to steel building frames. Tolerances in neither of these standards apply to buildings of mixed construction (for example, 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. Since the alignment of mixed construction members and encased members is not controlled by referencing the above standards, the engineer-architect should require that the location of all such materials contiguous to the precast unit be controlled within some stated limits. One recommendation is that the tolerances be no more than those specified in ACI 301 for reinforced concrete buildings. Should there be some doubt as to the appropriate magnitude of mixed construction tolerances, the precast concrete manufacturer may be consulted for advice.

3.7.4 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 above. The 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. adjustment in connections up to the 20th story and a 5 in. adjustment in connections above the 20th story if the engineer-architect insists on a true vertical plane. Adjustments of this magnitude in connections are not economically feasible. Therefore the precast concrete wall should follow the steel frame.

In determining tolerances, attention should also be given to possible deflections and/or rotation of structural members supporting precast concrete. This is particularly important for bearing on slender or cantilevered structural members. If the frame deflection is sensitive to the location or eccentricity of the connection, tolerances for location or eccentricity should be given. Consideration should be given to both initial deflection and to long-term deflections caused by plastic flow (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 the precast concrete and the support elements.

A structural steel frame presents different erection and connection problems from that of a concrete building frame. For example: structural steel cross sections, frequently relatively weak in torsion compared to concrete cross sections, generally require that the load be applied directly over the web or that the connection be capable of supporting the induced torsional moment. This in turn can require a stronger connection, as well as creating erection problems when the rolling tolerances of the steel beam approach their limits. When detailing precast elements for attachment to steel structures, allowance must be made in the precast element for sway in tall, slender steel structures with uneven loading, and deflections due to thermal effects.

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

The clearance necessary for erection of the wall will depend on the wall design, the dimensional accuracy of the building frame or other construction to which the wall is connected, and the limits of adjustment permitted by the connection details. If connections to the face of
a = Plan location variation from building grid datum ± 1 in.

\( a_1 \) = Plan location variation from center line of steel* ± 1 in.

b = Bearing elevation variation** from nominal elevation at support
   Maximum low 1/2 in.
   Maximum high 1/4 in.

c = Maximum plumb variation over height of element
   1/8 in. per 12 in. height
   1/2 in. maximum

d = Maximum jog in alignment of matching edges
   Architectural exposed edges 1/4 in.
   Visually noncritical edges 1/2 in.

e = Joint width variation from specified
   Architectural exposed joints ± 1/4 in.
   Hidden joints ± 3/4 in.
   Exposed structural joint not visually critical ± 1/2 in.

f = Bearing length*** (span direction) ± 3/4 in.

g = Bearing width*** ± 1/2 in.

*For precast elements erected on a steel frame, this tolerance takes precedence over tolerance dimension a.

**Or member top elevation where member is part of a frame without bearings.

*** This is a setting tolerance and should not be confused with structural performance requirements set by the engineer-architect.

Fig. 3.7-Erection tolerances for precast beams and spandrels required for proper interface with precast wall panels
spandrel beams or to columns are required, more clearance will be needed to install the fasteners than when the anchors are located on the top and/or bottom faces of beams and the sides of columns.

3.8-Clearances and tolerances for constructibility

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

- Between adjacent precast member: \( \frac{3}{8} \) in.
- Between precast and cast-in-place concrete: 1 in.; \( 1\frac{1}{2} \) in. preferred
- Between precast members and steel frame: 1 in.
- Between precast members and the frame of tall irregular structures: 2 in.
- Between precast column cladding and the column: \( 1\frac{1}{2} \) in.; 3 in. preferred

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

3.8.2 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. 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 some other value is specified by the engineer-architect, tolerances for such locating dimensions should be ±1 in. in all directions (vertical and horizontal) plus a slope deviation of no more than ±\( \frac{1}{4} \) in. for the levelness of critical bearing surfaces.

Connection details should provide for the possibility of bearing surfaces being misaligned or warped from the desired plane. Adjustments can be provided by the use of drypack concrete, nonshrink grout, or elastomeric pads if the misalignment from the horizontal plane does not exceed ±\( \frac{1}{4} \) in.

Where possible, connections should be dimensioned to the nearest \( \frac{1}{8} \) in. The minimum clearance between parts within a connection should not be less than \( \frac{1}{4} \) in., with \( \frac{1}{2} \) in. preferred. The minimum clearance or shim space between various connection elements should be a minimum of 1 in.

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 the 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 4-MATERIALS

4.1-Introduction

Basic materials used in the fabrication and erection of precast concrete wall panels are the same as those used in cast-in-place structural concrete. However, precast concrete wall panels also make extensive use of special materials, including exposed aggregates, admixtures, inserts and specialty coatings to enhance esthetic appearance. This chapter describes the following materials as used in precast concrete panel construction:

- Portland cement
- Aggregates, both standard and decorative for facing
- Admixtures
- Insulating materials
- Reinforcement and inserts
- Curing materials and sealers
- Joint sealants and fillers
- Surface retarders
- Form release agents

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

4.2-Portland cement

4.2.1 General — Usual practice is to use white, buff, or gray portland cement which meets ASTM C 150 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 it is important to take every precaution to assure that early concrete strengths are adequate.

4.2.2 Single source — On any given project, enough cement for the entire project should be procured from a single source so that 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 due to the cement. Total elimination of color variation is not possible since variables in other materials and in panel manufacturing may also have some effects.

4.2.3 Storage — Dry, covered storage areas should be provided for bulk or bagged cement. Bagged cement should be stored off the ground, preferably on wooden pallets and out of contact with 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 total height. Bulk cement should be stored
so that contact with tanks or walls where condensation can occur is minimized.

4.2.4 Sampling — 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, in case of problems with either strength or color uniformity.

4.3-Aggregates for structural or backup concrete

Normal weight or lightweight aggregates conforming to ASTM C 33 or C 330, respectively, should be used in backup or structural concrete for precast panels. Grading requirements for a backup mix may be waived if it is intended or necessary to provide a backup concrete with mechanical or physical properties similar to that of the facing or decorative aggregate concrete in order to minimize bowing or warping.

Aggregates for backup concrete should be stored in clean areas that are well drained and, if possible, in identifiable bins. The bins should be designed to avoid segregation, contamination or intermixing of different aggregates or aggregate sizes.

4.4-Facing aggregates

4.4.1 Grading

4.4.1.1 General - Uniform aggregates used for regular concrete are usually selected by standard sieve sizes to provide a balance of both fine and coarse sizes. The ideal grading is one that combines aggregate sizes to produce the maximum weight of aggregate per unit volume of concrete. Most concrete mixes are chosen with this in mind but are often limited on the upper end of the coarse aggregate size by:

1. The dimensions of the panel to be cast
2. Clear distance between reinforcement and the form surface
3. Clear distance between the reinforcement and the form surface
4. The desired finish.

4.4.1.2 Gap grading of facing aggregates - Since precast concrete panels frequently use exposed aggregate, the desired surface finish, appearance, and texture frequently dictate the grading of both the fine and coarse aggregates. Gap grading may be used to 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 also select tighter or more restrictive gradings in an attempt to improve uniformity. Common sizes of gap-graded fine aggregates are 30 to 50 mesh and 16 to 30 mesh. The use of the coarse and fine sizes combined can produce a gap-graded combination that results in less segregation and a more uniform surface finish.

4.4.1.3 Schedule of sizes — Table 4.4.1.3 shows four different size gradings established by precast industry suppliers of aggregates for use in exposed aggregate precast concrete. However, 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. and smaller, may pull out of exposed aggregate finishes during surface finishing.

4.4.2 Types and quality of facing aggregates

4.4.2.1 General - Decorative facing aggregates are normally used only in the exposed panel faces because of cost. The thickness of the face layer depends on the size of aggregate, but it should be thick enough to prevent the backup concrete from showing on the exposed face. The face concrete thickness should be 1.5 times the maximum size of coarse facing aggregate but not less than 1 in.

Aggregates for facing mixes should be stocked in sufficient quantities from the particular source to complete the entire project. Failure to plan appropriately may cause unwanted changes in color or texture.

4.4.2.2 Specific surface color and texture - Special aggregates selected for facing use include naturally occurring aggregates such as selected gravels, granites, traprock, marble, limestone, and quartz, quartzite, feldspar and obsidian. Selection should be based on performance of the facing aggregate in approved panel samples. Approval should be based on both manufacturing and esthetic acceptability.

4.4.2.3 Durability concerns - Some limestones, marbles, and sandstones are not durable on exposed exterior surfaces. All facing aggregates should have proven service records or be shown to be acceptable under laboratory test conditions before being used in precast panels. Appropriate tests include petrographic examination and expansion tests (ASTM C 227).

Facing aggregates that have passed laboratory durability testing or have good service histories rarely have problems with alkali-aggregate reactivity. If such a reaction is suspected from a new or unknown combination of aggregates and cement, the aggregate should be examined petrographically, and expansion should not exceed ASTM C 33 limits. If the limits are exceeded, it is recommended that a low alkali cement, with a maximum of 0.6 percent Na₂O equivalent according to ASTM C 150 or a material that has a proven record to prevent harmful expansion, that is, fly ash, be used with that aggregate. Occasionally materials that have been shown to prevent harmful expansion, such as fly ash, may be used if the matrix color meets architectural appearance requirements.

4.4.2.4 Staining — Occasionally, coarse facing aggregates may contain particles with an iron content high enough to result in unsightly stains. This characteristic usually shows up at a later date in finished panels due to oxidation from exposure to the atmosphere. Selectivity by the panel producer and a good working knowledge of aggregate materials and their service records are currently the only assurance against long-term iron stains from aggregates. Test for the quantity of iron bearing particles in an untried aggregate should be made according to ASTM C 641 and the aggregates should show a
Table 4.4.1.3—Typical industry size specifications for exposed aggregate

<table>
<thead>
<tr>
<th>Sieve opening</th>
<th>Size D 1⅜ to ⅜ in.</th>
<th>Size C ⅛ to ⅜ in.</th>
<th>Size B ⅛ to ⅜ in.</th>
<th>Size A ⅛ to ⅜ in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>in.</td>
<td>mm</td>
<td>(35 x 22 mm)</td>
<td>(22 x 13 mm)</td>
<td>(13 x 6 mm)</td>
</tr>
<tr>
<td>1½</td>
<td>38</td>
<td>100</td>
<td>95-100</td>
<td></td>
</tr>
<tr>
<td>1⅛</td>
<td>35</td>
<td>95-100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>25</td>
<td>30-60</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>⅛</td>
<td>22</td>
<td>20-40</td>
<td>95-100</td>
<td></td>
</tr>
<tr>
<td>⅜</td>
<td>16</td>
<td>0-10</td>
<td>30-50</td>
<td></td>
</tr>
<tr>
<td>⅜</td>
<td>13</td>
<td>10-20</td>
<td>95-100</td>
<td></td>
</tr>
<tr>
<td>⅛</td>
<td>9</td>
<td>0-10</td>
<td>40-70</td>
<td></td>
</tr>
<tr>
<td>⅛</td>
<td>6</td>
<td>5-20</td>
<td>95-100</td>
<td></td>
</tr>
<tr>
<td>⅛</td>
<td>3</td>
<td>1-10</td>
<td>15-35</td>
<td></td>
</tr>
<tr>
<td>⅛</td>
<td>9</td>
<td>0-10</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4.4.2.5 Glass or ceramic aggregates—Glass or ceramic aggregates that may be used for bright color or for special effects should be nonreactive with the cement used. The “quick chemical test” in ASTM C 289 may be used for detection of glass or ceramic aggregates which are reactive. Ceramic aggregates may exhibit brittleness and breakdown during casting. Glass aggregates have low absorption and good durability, but have the disadvantage of low compressive strength and low bond strength with the cement paste. Production testing of glass and ceramic faced panels is highly recommended.

4.5-Admixtures

4.5.1 General—Chemical or mineral materials may be added to the concrete mix to bring about specific changes in the mix properties. ACI 212.3R contains recommendations for the use of chemical admixtures, including limits on chloride content of hardened concrete (see also Section 4.5.3). For protection of reinforcement from corrosion, ACI 222 recommends limits the acid-soluble chloride ion content in hardened concrete. All prestressed concrete and any reinforced concrete exposed to moisture or chloride in service falls into one category and any reinforced concrete that is dry or protected from moisture in service falls into the other category.

4.5.2 Air-entraining agents—Air-entraining agents should be used in all concretes that may be exposed to freezing and thawing cycles when saturated with water. The added protection against freeze/thaw deterioration far outweighs any loss of strength or density. A “normal” dosage of air-entraining agent, the amount that will provide about 9 percent air in the mortar fraction of the concrete, is recommended. Because of the unusual nature of most facing mixes, a specification for the amount of air-entraining admixture rather than a fixed percentage of air is recommended. Refer to ASTM C 260 and C 185.

4.5.3 Mineral admixtures and pozzolans—On rare occasions where a particularly smooth surface is desired, the addition of fine minerals or pozzolans may be made to the mix. The typical curing period for precast panels is often too short to allow pozzolanic action for increased strength.

4.5.4 Accelerating admixtures—Accelerating admixtures, ASTM C 494 Types C and E, reduce concrete setting time and produce rapid early strength gain which can aid in panel casting operations. Rapid strength gain may also be accomplished with higher cement content, use of Type III high early strength cement, heated water and aggregates, or by steam curing. Accelerators containing calcium chloride or thiocyanate ions may contribute to corrosion of reinforcement. Calcium chloride also affects color. The committee recommends that accelerators containing more than 0.1 percent calcium chloride be used only when it can be demonstrated that they neither initiate nor promote corrosion of steel in any precast panel, or adversely affect color of the panel.

4.5.5 Retarding admixtures—Chemicals to retard the set of concrete, ASTM C 494 Types B, D and G, are normally not used in precast concrete wall panels except in hot weather. Retarders delay the time of set and allow longer finishing time. They generally do not fit into a high speed casting operation.

4.5.6 Water reducing admixtures—Water reducing admixtures, ASTM C 494 Types A and F, are used in precast concrete wall panels where it is desirable to reduce the bleed water or to increase the workability of the concrete without adding water. This group includes high range water-reducers (super-plasticizers) for conditions where placing concrete is difficult. Laitance, bleeding and efflorescence can be minimized by reducing water requirements. Water reducers may be helpful in harsh mixes or where gap-graded aggregates are being used. Water reducing components must meet the requirements of ASTM C 494 Type A, and should be checked for compatibility with the cement and with any air-entraining admixture to be used.

4.5.7 Coloring materials—Both pigments and dyes are used to enhance the color tone of concrete in precast panels. It is important to have tests or performance
records that reliably indicate the color stability of any coloring agent. Experience shows that there is generally poor color stability with organic blacks, blues, and greens.  

4.5.7.1 Pigments — Pigments commonly used to color concrete are finely ground natural or synthetic mineral oxides. Synthetic oxides are usually more satisfactory since they are manufactured in more shades, have more consistent properties, give better color intensity and have longer permanence. Synthetic pigments may possibly react with other products used on concrete facing mixes such as retarders or muriatic acid. All pigments should be tested prior to use and should conform to ASTM C 979. Iron oxides produce shades of yellow, buff, tan, brown, red, maroon, and black. Chromium oxide produces green shades. Cobalt oxide produces shades of blue. Varying amounts of these oxides, added as a percentage of the cement content by weight, produce various shades. Amounts in excess of 5 percent by weight of cement seldom further increase color intensity. Amounts greater than 10 percent may adversely affect concrete quality and are not recommended. Pigments used with white cement will produce clearer and brighter shades than if used with gray cement. Dry mixing of the pigment with the cement prior to concrete mixing is preferred. Some cement manufacturers can provide premixed or pigmented cements.  

4.5.7.2 Dyes — Organic phthalocyanine dyes have been successfully used to produce light to dark shades of blue and green in concrete. While their cost per pound is high, they are used in quantities of less than 1 percent by weight of cement and can be dispensed in the mixing water, eliminating the need for preblending. Although certain organic phthalocyanine dyes work well, others may fade quickly upon exposure to sunlight.  

4.6-Insulating materials  
A wide variety of insulating materials is available to provide the desired thermal properties for sandwich wall panels. Since thermal conductivity usually varies with density of insulating material, the unit weight is used to classify insulating materials as follows: a) Density of 15 lb per cu ft or less

- Plastic materials such as polyurethane foam boards;
- polystyrene foam boards or granules
- Glass materials including foamed glass boards or granules; glass fiber batts
- Paper materials such as paperboard honeycombs filled with insulating granules or aggregate; cellulose granules.
b) Density of 16 lb per cu ft to 50 lb per cu ft

- Foamed concrete: autoclaved cellular concrete boards; nonautoclaved cellular concrete boards of granules
- Mineral aggregate concrete: vermiculite concrete boards or granules; perlite concrete boards or granules.

Being very porous, many of these insulating materials will have high initial rates of water absorption and can absorb water from the fresh concrete placed over and around them. Glass batts and granules should be enclosed in plastic bags to prevent absorption and rapid drying of the surrounding concrete. Open-cell board insulation should have a waterproof membrane coating applied before use.  

4.7-Reinforcement  
Reinforcement for precast panels includes prestressing materials, deformed bars, and welded wire fabric. Reinforcement also includes those ribs or metal shear ties used in three-layer sandwich panel construction to connect the two outer layers of concrete. The shear ties may be made of expanded metal and are commercially produced as masonry reinforcement or building studding. Metal shear ties generally incorporate diagonal members for proper resistance to horizontal shearing deformation in sandwich panels. Some sleeve anchors do not have diagonals but may still be acceptable.  

4.7.1 Deformed reinforcing bars — Deformed reinforcing bars are manufactured by hot rolling deformations onto steel and are made in accordance with ASTM A 615 (billet steel), ASTM A 616 (rail steel) ASTM A 617 (axle steel), and ASTM A 706 (low-alloy steel). Bars are normally used in straight lengths but can be bent to form hooks required for anchorage purposes. ASTM A 496 presents requirements for deformed wire used as concrete reinforcement.  

4.7.2 Welded wire fabric — Welded wire fabric (WWF) is available in a wide variety of mesh spacings and wire gauges with both plain and deformed wire being used. WWF should be manufactured in accordance with ASTM A 185 and A 497 for plain and deformed wire, respectively.  

4.7.3 Prestressing materials — Steel wire, bar, and strand for prestressed concrete should meet requirements of ASTM A 416, A 421 and A 722.  

4.7.4 Corrosion protection of reinforcement — When esthetic considerations cause reduction of the concrete minimum cover below that ordinarily specified or recommended, reinforcement in thin precast concrete panels (under 4 in.) may be susceptible to corrosion. In such cases, the long-term appearance and durability of the panels may require corrosion protection for reinforcing materials or the use of stainless steel reinforcement or other reinforcement clad with copper or other metals less likely to corrode than uncoated reinforcing steel.  

4.7.4.1 Galvanizing — Galvanized welded wire fabric is readily available and the cost premium is relatively low in comparison to the unprotected product. Galvanized reinforcing bars are not as readily available and the extra cost may be substantial. Use of galvanized bars may be minimized or avoided by proper design, maintenance of minimum cover, and manufacture of the panels so that bar location, concrete placement, and consolidation are precisely controlled. Galvanizing procedures should conform to ASTM A 767 and A 153, including supple-
mentary requirements.

4.7.4.2 Epoxy coating — Epoxy-coated reinforcing bars (ASTM A 775) and welded wire fabric (ASTM A 884) have been used extensively in severe exposure environments, but only minimally used in architectural precast panels. The report by ACI 222 discusses the corrosion mechanism and corrosion protection in detail. Development length must be increased for epoxy coated bars as required by ACI 318, Section 12.2.4.3.

These bars are very resistant to corrosion if the coating is uniform. Bars coated when straight and subsequently bent have shown that bending has no effect on the coating integrity. If the coating is damaged of non-uniform, the bars have to be touched up with commercially available epoxy compounds to prevent serious corrosion. Bar tying should be done with nylon or plastic coated tie wire rather than black wire. Bar supports should be stainless steel, epoxy coated or solid plastic.

4.7.4.3 Other coatings - Other coatings available for corrosion protection include various paints such as inorganic zinc-rich types, epoxy paints, and certain proprietary chemical compounds which combine with oxide coatings to form a protective layer. These materials may be brush, bath, or spray applied. In evaluating these coatings, known performance characteristics and test data should be considered.

4.8-Inserts and miscellaneous hardware

Inserts are items cast into the panel for lifting, holding, or attaching the precast panel to other structural members. Installing inserts after casting by drilling into place is not recommended unless something happened to the cast-in inserts and a field solution is required.

Such items as channel sections, framing, studs, anchors, expansion anchors, and inserts should be made from materials that are permanently ductile. When reinforcing bars are used as anchors or inserts, precautions (Section 5.4.2.3) should be followed to ensure adequate strength and ductility when their connections are welded. Brittle materials such as grey-iron castings should not be used. Specifications for bolts include ASTM A 307, A 325 and A 490. Specifications for stud welded anchors include ASTM A 108 and A 496.

4.8.1 Expansion anchors — Expansion anchors should conform to applicable bolt specifications and be performance tested in accordance with ASTM E 448. All items should have documented chemical and physical properties and be used in accordance with the manufacturer’s recommendations and/or test data.

4.8.2 Corrosion protection — Where corrosion protection is required for embedded or exposed hardware, noncorrosive materials such as stainless steel, in accordance with ASTM A 276, or the hardware may be protected with a coating such as zinc, cadmium, epoxy, or corrosion resistant paint may be used. Care must be taken so that the protective coatings do not interfere with subsequent fit of the nuts onto threaded portions of the fasteners. Hex nuts and washers, or other matching hardware used with exposed insert connections, should be zinc or cadmium plated. The use of hex lock nuts with nylon locking washers is suggested.

4.9-Curing materials and sealers

4.9.1 Curing materials - Although not generally used, curing compounds are preferred over water, burlap, or other wetted coverings where additional curing of the concrete is required. A wide range of curing compounds is commercially available; they should be supported with test data and user experience before acceptance. Steam curing is discussed in Section 5.7.3.2. Curing compounds and sealers may have to be removed if the panel surface is to be painted.

4.9.2 Surface sealers - The use of clear, protective, water-repellent sealers, often also referred to as coatings, to maintain panel appearance remains an area of controversy with panel producers. A justification for use of sealers is the potential improvement of weathering qualities in urban or industrial areas. A sealer may reduce attack of the exposed concrete by airborne industrial chemicals. Laboratory exposure tests and long time outdoor exposure plots have yielded a wide range of results. Some sealers produce severe discoloration within an exposure period varying from one week up to several months. Panel surfaces that have been sealed may discolor because: (a) some sealers attract hydrocarbons to the surface of the sealer; (b) some sealers have little resistance to discoloration by the sun’s ultraviolet rays; and (c) other sealers may be affected by temperatures of 145 F or above.

Tests have shown that sealers do not improve resistance to freezing and thawing. Freeze/thaw durability is best achieved with air-entraining agents as outlined in Section 45.2.

The use of sealers should be based on prior experience and a careful study of test data for conditions of similar exposure. They should be applied in strict accordance with the manufacturer’s recommendations. Sealers generally should not be applied on surfaces that will be in contact with joint sealants.

4.9.2.1 Silicone sealer performance — Surfaces treated with silicone formulations vary widely in performance. The service life of silicone sealers is controversial and probably shorter than advertised. Experience with silicone sealers indicates that they should not be used on exposed quartz aggregates, and that durability results with use on other aggregates are marginal. In urban areas, some silicone sealers attract airborne hydrocarbons resulting in premature discoloration of white or light colored panels within a short period.

Silicone sealers interfere with the bonding of patches and prevent the bonding of joint sealants. If used, silicone-based materials should be applied only after patching and joint sealing are completed. Committee 533 does not recommend the use of silicone-based sealers.

4.9.2.2 Other sealers — Better results have been ob-
tained with methyl methacrylate forms of acrylic resin on exposed aggregate surfaces. Satisfactory results have also been achieved with other acrylic copolymers, silanes and siloxanes.

A wider range of acceptable sealers is available on less delicate surfaces such as plain or ribbed concrete panels. Sealers that do not perform well on exposed aggregate surfaces or very light colored surfaces are often acceptable on plain or ribbed concrete.

4.10-Joint sealants and fillers

4.10.1 Mortars - Cement mortars are not extensible and cannot accommodate panel movement. Although they are not suitable as sealants, mortars may be used for packing joints in connections in combination with other sealers. Mortars are ideal for the base of load-bearing panels supported by shims.

4.10.2 Elastomeric sealants - Only elastomeric materials should be used as sealants in precast panel installation. Elastomeric sealants (also referred to as caulks) include polysulfides, silicones and urethanes. They may be one- or two-part compounds but either is to be preferred over oil base types. Despite higher initial cost, elastomeric sealants are preferred because of lower maintenance costs, better weathertight joints, and longer life. Consult ACI 504 and ASTM C 962 for detailed information.

Nonstaining elastomeric type joint sealants should be selected to prevent the possibility of bleeding and heavy dirt accumulation. High performance one- or two-part sealants such as polysulfides, urethanes, silicones or other sealant material are recommended for weatherproofing joints in precast panels. These sealants should withstand joint movements of at least ± 25 percent. If greater sealant movement capacity is required, consult with manufacturers of low modulus sealants. The sealant selected should match as closely as possible the color of the precast panel. This will reduce the visual effect of variations in joint dimensions.

4.10.3 Joint fillers - Backup fillers are needed in joints to control the depth of the sealant, to facilitate tooling of the sealant, and to serve as a bond breaker to prevent the sealant from bonding to the back of the joint. The filler material should be nonstaining to the sealant. Asphaltic (bitumastic) fillers should not be used. The sealant manufacturer can advise which filler materials would be compatible with the selected sealant. The recommended shape factor should be listed.

Acceptable fillers are those which compress into the joint and respond to panel movement. A round filler profile provides maximum edge area with minimum cross section for best sealant adhesion. The best filler profile is a rod of spongy or foamed material that is closed cell to prevent moisture retention. If a stiff filler material such as cork, wood, hard rubber or concrete mortar must be used, a strip of polyethylene sheet or similar material is recommended to break the bond between the filler and sealant.

4.11-Chemical retarders

4.11.1 General - Chemical retarders are specialized chemicals which temporarily delay the cement paste from hardening at the surface of the precast panel. After the base concrete of the panel hardens (normally overnight), the retarded cement paste is removed by brushing, high pressure water washing, or sandblasting to expose the aggregates. Brushing or water washing will not change the natural look of the aggregates, but sandblasting may dull the surface.

Two types of chemical retarders are used in precast panel fabrication. Form retarders, usually fast drying solvent-based materials, are applied to the form surface. These retarders are designed to resist the abrasion inherent in the placement of concrete. Surface retarders are water based materials applied to the top surface of freshly placed concrete. They are usually sprayed on with garden-type spray applicators. Before the retarder is sprayed on, additional aggregate may be placed on the surface and troweled in to provide a more uniform finished surface.

4.11.2 Depth of reveal - Form and surface type retarders are available to etch the concrete to different depths allowing for design flexibility. As a general rule, the retarder should expose not more than 40 percent of the diameter of the aggregate at the surface. Retarders can be used to produce finishes from the lightest reveal which just removes the surface skin to deep reveals using aggregates up to 1 1/2 in.

4.12-Form release agents

4.12.1 General - Modern release agents are formulated from a variety of ingredients to perform several functions. Their primary purpose is to release the panel (aid in debonding) from the form. Other functions include, minimizing or eliminating bug holes and stains, minimizing form clean up time, keeping cementitious materials from building up on the form facing, not interfering with the bonding of construction and/or architecturally esthetic materials to the hardened concrete surface, not degrading (and thereby causing stains) form facing materials, not staining concrete when steam curing is used, contributing towards the production of high visual impact concrete surfaces and being easy to apply in all seasons.

4.12.2 Chemically active release agents - Chemically active release agents are the most common type. Their releasing ability is due to the chemical reaction of free lime from the fresh cement paste with chemicals in the release agent coating the form surface. This chemical reaction produces a slippery, water-insoluble soap or grease, which provides for easy form removal. Typically the chemically active ingredients are fish oils, vegetable oils, animal fats, or combinations thereof.

4.12.3 Emulsion type agents - Some release agents use water emulsions for a carrier instead of petroleum derived oil. Some emulsions are chemically active agents while others facilitate form release by producing a barrier
film, much like fuel oil does. Generally, emulsion type release agents will not harm any of the form facing materials that would be sensitive to petroleum derived oils. Cold weather operations require storage and use considerations different from other release agents.

4.12.4 Petroleum derived agents - Release agents made entirely from petroleum derived oil, that is, fuel oil, kerosene, etc., function by producing a barrier between the form face and the concrete. This barrier type release agent generally causes more bug holes, staining, and poorer form release than chemically active types.

4.12.5 Application of release agents to formwork - Release agents should be applied in a thin uniform coating on clean dry form facings. Usually this is done by spraying. The release agent should be applied in a manner and schedule so as to avoid coating the reinforcement.

CHAPTER 5-PANEL FABRICATION AND DELIVERY

5.1-General requirements

5.1.1 Preparation of design calculations and production and erection drawings - The precast manufacturer prepares erection and production drawings for the precast panels, complete with all necessary details for the fabrication, handling and erection of the precast products. In order to do this the manufacturer should have all applicable contract documents, including specifications, architectural, site and structural drawings. Erection drawings and hardware from other trades must be provided within contractual schedules. Detailing methods vary with the manufacturer; however, elevations and horizontal dimensions should be shown which locate and mark each precast element and give its relationship to windows, openings, and adjacent building components. Details should provide size, shape, dimensions and profiles of each member. Connections, reinforcement, and individual mark numbers should be shown. Erection drawings should show the following: (a) proposed sequence of erection, if required; (b) location and details of hardware embedded in or attached to the structural frame; (c) method of plumbing (adjusting the vertical orientation) panels and adjusting connections and (d) handling loads and additional reinforcement due to transportation and erection stresses. Joint and joint sealant details should be shown where applicable. Further, special fittings such as stripping, lifting or erection inserts, anchoring details, reglets, cutouts, pipe sleeves, other embedded items and openings should be carefully located and dimensioned.

Drawings and calculations prepared to show the above should be forwarded to the general contractor and the engineer-architect for approval as recommended in Section 1.3.4.

5.1.2 Manufacturing facilities - Facilities for the production of precast panels vary widely. Production facilities will be affected by the size, weight, and volume of the products produced and by the climate and proximity of marketing areas. At times the requirements of a specific project warrant casting on the job site. A site precaster faces a few more problems than a plant precaster such as: lack of tightly controlled batching conditions and less than ideal curing and protection from the elements; possible difficulty of obtaining a skilled labor force; and possible lack of management or supervisory group experience in precasting operations. Recommendations in this guide will help to overcome these possible deficiencies. The manufacturing facility, whether at the site or in a plant, should adequately provide the following:

- Facilities to receive and store raw materials such as cement, aggregates and reinforcing steel
- Facilities for controlled proportioning and mixing of concrete
- A covered area for manufacturing of molds and forms
- An area for assembly and fabrication of reinforcement
- An enclosed or covered area (depending on the climate) for the casting operations (see Fig. 5.1.2)
- Additional space for the finishing and curing operations
- Adequate space for convenient and proper storage
- Equipment capable of lifting and handling panels of the size and weight to be manufactured
- Facilities for prestressing the precast wall panels, if required

5.1.3 Production and storage areas - Facilities for batching and mixing concrete should be in accordance with ACI 304, providing for accurate batching of aggregates, cement, admixtures, and water. Equipment should be available to determine the amount of free moisture in the coarse and fine aggregates. Moisture compensation based on devices using conductivity is known to vary with the density of the aggregates and is not recommended. Facilities should be provided and monitored to prevent frozen aggregates being introduced into the concrete. Mixing equipment should be adequate for the size of the operation and capable of thoroughly and uniformly mixing the concrete ingredients. Panel production areas should be protected against rain, wind, dust, and direct sunlight and have heat control to prevent concrete temperatures from dropping below 50 F. Panel storage areas should afford easy access and ready handling of the stored units. The surface should be clean, hard, level, and well-drained to permit well-organized storage, and to minimize or prevent warping, bowing, chipping, cracking, discoloration, staining or soiling of the precast panels.

5.2-Molds (forms)

5.2.1 General - Wood, concrete, steel, plastics, plaster, polyester resins reinforced with glass fibers and combinations of these have all been used successfully as
a mold or form material for precast panels. Various patterns made of rubber, pressed metal, or vacuum-formed plastic may be combined with the basic materials for special effects. For complicated details, molds of plaster, gelatin, or sculptured sand have been combined or reinforced with wood or steel, depending on the size of the panel to be cast. See Figs. 5.2.1.1 and 5.2.1.2.

Typically the panel producer selects the proper mold, based on considerations of cost, maintenance, method of consolidation, reuse, details of the panel, possible salvage and on the desired finish and texture of the product. Where the engineer-architect requires a special mold or finish or a particular mold material, these requirements must be clearly set forth in the contract documents.
5.2.1.1 Dimensional stability and integrity - All molds, regardless of material, should conform to the shape, lines and dimensions of the precast panels to be produced. They should be sufficiently rigid to meet the casting tolerances recommended in Chapter 3. Molds should be sufficiently tight to prevent leakage of mortar or cement paste, and should be designed to prevent damage to the concrete from: (a) restraint as the concrete shrinks; (b) the stripping operation when the unit is lifted from the mold; and (c) dimensional changes during prestressing. For prestressed (pretensioned) units it may be desirable to design the molds strong enough to support the prestressing force (i.e., self-stressing forms).

In molds longer than 20 ft, allowance for shrinkage and thermal expansion or contraction should be considered in the design of the master pattern and/or the mold. Master molds (described in detail in PCI Architectural Precast Concrete) are sometimes used to cast panels of several different designs by pre-engineering a number of mold adjustments (see Fig. 5.2.1.3).

5.2.2 Steel molds - Steel molds are often selected for precast members when it is anticipated that numerous assemblies and disassemblies of the mold will be required. Properly designed steel molds have great potential reusage; they need be discarded only when damaged, or when they show surface imperfections from drilling for changes or from alteration. Steel molds should be well braced and examined for bulging or buckling. Dimpling, twisting, or bending may occur if the form surface is not properly stacked for storage.

When steel plates are used for the base of the mold, it is desirable to use a single piece which has been “stretcher-leveled” in the steel plant. Joining of two or more steel plates by welding to form a flat surface is difficult due to distortion from the heat of the welding operation. If joining is required, the welds should be ground smooth and coated with an epoxy or similar material adequate to hide the joint imperfections. A test section should be cast at the joint to determine that the joined area can produce an acceptable finished product.

If a prestressing force is to be applied to the form, the self-stressing form must be strong enough to resist the prestressing force without buckling or wrinkling.
5.2.3 *Concrete molds*—Concrete can be formed into practically any shape and has excellent rigidity, dimensional stability, and the potential for a large number of reuses. Concrete molds are manufactured by casting over a master model fabricated to very close tolerances. This model may be used for the production of a series of identical molds. Frequently concrete molds are treated with epoxy, plastic resins, or paraffin wax to reduce mold repair and to improve their release capability. These resins and other coatings, render the concrete nonabsorbent and produce a more uniform finish on the precast product. Concrete molds may also be adapted to become a self-stressing form for use in prestressing a concrete panel. When concrete molds are used, adequate draft should be provided on all surfaces in the direction of stripping or removal, so that the product may be easily lifted without damage to either the panel or the mold.

5.2.4 *Wood molds*—Wood molds vary from simple wooden molds (particularly applicable when relatively small, flat panels are being produced) to elaborate, complicated molds of unusual shapes and large dimensions. Molds of wood should be treated to prevent excessive absorption, thus contributing to uniformity of panel finish. The treatment also tends to stabilize the form dimensions.

Periodic renovation of wooden molds is necessary, and special care should be exercised to ensure that multiple use does not cause the mold to swell or bulge. Mold dimensions should be checked after each use. Wood molds should not be used if steam curing is planned before stripping. Craftsmanship and joiner-y should be of high quality to achieve joints which are not objectionable in appearance. The joint location should be preplanned, subject to approval by the engineer-architect.

5.2.5 *Plastic molds*—Fiber reinforced plastics produced from polyester or epoxy resins have considerable application because they can be easily molded into complex shapes and can impart a great variety of patterns to the finished product. Properly designed plastic molds have excellent performance and reuse expectancy, and the surface may be renovated as needed.

Fiber reinforced plastics have a fairly low modulus of elasticity and are somewhat flexible, even though they have excellent tensile strength. If the mold is made entirely of plastic, it should be well supported along edges and flat areas. This may be accomplished by reinforcing it with lumber, steel shapes, or other materials as an integral part of the mold construction. This type of mold is frequently designated a “mold liner,” and requires a “mother” mold to provide adequate support.

The high gloss finish imparted to a precast panel by some mold surfaces and/or mold liners (generally plastic) may be desirable for indoor decoration. When used for exterior work however, the gloss may disappear, usually in a nonuniform manner, because of weathering. Crazing and other minor surface imperfections may be more apparent on this type of surface. A high gloss finish should be avoided on exterior work, unless the panel producer can show successful installations of this finish in a similar climate.

5.2.6 *Form liners*—Textures ranging from muted expression to bold relief are obtained with different types of form liners (see Fig. 5.2.7). Draft must be considered for all types of liners to prevent chipping or spalling during stripping. Rubber matting is an effective liner, reproducing complex patterns faithfully on the concrete surface. While rubber is generally satisfactory, it should be tested for possible staining or discoloration of the concrete. Trial castings will also determine the most effective time for stripping to ensure that the surface remains intact and that the liners can be reused.

Wood liners, either boards, plywood panels, or nailed-on inserts, work well. Wood liner surfaces should be sealed to prevent moisture absorption and then lightly coated with a form release agent before casting.

Plastic sheets may be vacuum formed to provide varied patterns on either textured or glossy-smooth concrete surfaces. The extremely fine finish of plastic formed concrete enhances the attractiveness of integral colors and because of high reflectivity, smaller amounts of pigment are required to obtain a given color intensity. Glossy and smooth surfaces are best for indoor rather than outside exposure.

Polyethylene film laid over uniformly distributed cobblestones provides dimpled surfaces. Pieces of polystyrene foam, shaped and attached to the form, leave deeply impressed designs after removal from the concrete face.

5.2.7 *Verification and maintenance*—Molds should be checked in detail after construction and before the first unit is made. A complete check of the first product from the mold further verifies the adequacy of the mold. Checking of the mold by evaluating the product is feasible only where an element can positively be identified with the specific mold in which it was cast.

Molds should be cleaned between castings and kept in good condition to provide a uniform product of high quality. Steel forms must be carefully maintained to avoid discoloration of the concrete from iron oxides. Joints may open, weldments come loose, rubber sealing strips erode; wood loses its protective coating, absorbs moisture and may swell or warp; and side rails bow due to daily production. It is therefore necessary to check the molds regularly, at least once a week, for their soundness, surface, and dimensional stability, and to repair damage before it affects the quality of the product.

Forms should be reassembled within the dimensional limitations specified for the product on the shop drawings. The squareness of the form should be checked by comparing diagonal measurements between the corners of the form. Bulkheads, templates, and similar equipment having influence on the accuracy of dimensions and alignment should be regularly inspected and maintained. If more than one form is used to produce a given unit, a comparative dimensional check should be made before casting of the initial precast concrete panels.
Fig. 5.2.7-Form liner relief patterns

All positioning holes or slots holding any cast-in materials in a given position should be checked to ensure that continuous mold use will not create wear and exceed acceptable tolerances. When using wood molds, these clamp-on areas should be protected with corner plates.

5.3-Concrete proportioning and mixing

5.3.1 Introduction — Information on mix proportions should be recorded and kept at the concrete batching plant. Although the same technology used for making cast-in-place concrete must be considered in precasting, specific mix proportions and mixing procedures differ from those for conventional concrete, particularly in the emphasis on finish and durability of the concrete surface. Precast manufacturers usually design and control the concrete used for the precast product because of the factors listed below. Before a concrete mix can be properly proportioned, several factors must be considered:

- The finish, size and shapes of units to be cast
- The method of consolidation; must be known to determine the required workability
- The maximum size of the coarse aggregate
- The required compressive strength
- The required surface finish as it affects the ratio of coarse to fine aggregate
- Exposure to severe weather or environmental conditions

5.3.2 Water-cement ratio and consistency (slump) — With given quantities of cement and aggregate along with proper curing, the quantity of mixing water determines the strength of the concrete. Water should be held to a minimum to prevent a substantial decrease in strength and durability. Excess water in the facing and/or backup mix may be removed before initial set by a vacuum process, application of hygroscopic materials or low-slump mixes.

One of the most important factors affecting color uniformity is variation in the water-cement ratio. The water-cement ratio must be consistent from batch to batch from the beginning of the project to the end or the possibility of precast concrete panel rejects increases because of lack of color uniformity. Very stiff mixes using a minimum of cement and only enough water to hydrate the cement are economical only in terms of material costs. These mixes generally require more placing labor, and these added placing costs may more than offset any material cost savings.

Concrete mixes should always have a consistency and workability suitable for the conditions of the project. Heavily reinforced thin sections require more plastic mixtures than large members with little reinforcement. High-range water-reducers can increase workability while maintaining a relatively low water-cement ratio. It is important that the specifier realize that even with the same proportions, the slump may vary with changes in climactic conditions and normal variations in materials. Concrete mixtures using natural sand and gravel aggregates require less water for workability than do concrete mixes using crushed sand and crushed coarse aggregate. The shape and size of the aggregate will affect water demand, which in turn affects the water-cement ratio.

5.3.3 Proportioning - Concrete mixes should be proportioned in accordance with ACI 211.1 and ACI 211.2 to produce a specified compressive strength of at least 5000 psi measured at 28 days on standard 6 x 12-m cylinders. Some decorative aggregates have characteristics that do not permit attainment of compressive strength. Caution is advised in selecting such aggregates.

The objective of proportioning is to achieve a practical combination of materials that will provide the required
qualities of the hardened concrete at an economical cost. Concrete mixes should be proportioned and/or evaluated for each individual project with respect to strength, absorption, and resistance to freezing and thawing as appropriate to the intended environment.

For normal weight aggregate concrete mixes, the ratio by volume of fine aggregate to coarse aggregate usually is on the order of 1:3 for facing mixes, whereas standard mixes are usually in the range of 1:1 to 1:2 or 1:3. For lightweight aggregate concrete mixes, the volume ratio may vary depending on the type of lightweight aggregate used. The aggregate producer should be consulted about specific material characteristics and recommended mix designs. The lower ratio of fine to coarse aggregate results in a more uniformly textured finish caused by a maximum concentration of coarse aggregate in the facing mix. All mixes for panels exposed to freeze-thaw conditions should include entrained air for increased durability.

5.3.3.1 Facing and backup mixtures - Differing aggregate gradings and mix proportion designs for facing mix concretes make it impossible to specify a given percentage of air for these mixes. Consult the PCI Manual for Quality Control for Plants and Production of Architectural Precast Concrete Products for added information.

Standard mixes may be used as backup mixes if the physical characteristics are similar to the facing concrete. Where a precast unit consists of a face and backup mix, the mixes should have reasonably similar shrinkage, thermal coefficients of expansion and modulus of elasticity to avoid undue bowing or warping. Consequently, these mixes should have similar water-cement and aggregate-cement ratios. The combination of a normal weight face mix and a backup concrete with lightweight aggregates may increase the possibilities of bowing or warping.

5.3.4 Mixing - The mixing procedures used in the manufacture of precast wall panels vary due to the variety of equipment and methods of panel manufacturing.

To maintain batch-to-batch uniformity, materials should be properly sequenced and blended during charging of the mixers. Good mixing practices include the following:

- The mixer should be operating while all materials are being charged.
- All admixtures should enter the mixer with the water and be charged into the mixer at consistent times in the mixing sequence. Batching air-entraining agents simultaneously with water reducers or retarders can cause the combination to gel.
- Pigments should be preweighed and batched from packages that are of a size appropriate for a single batch. The aggregates and cement should be introduced together.
- Where mixers of 1 cu yd capacity or less are used, the aggregates may be placed into the mixer first and then the cement and water introduced together.

5.3.4.1 Mixing time - After all materials have entered the mixer, they should be mixed for a minimum of one minute, or as recommended by the mixer manufacturer, until all ingredients are thoroughly distributed and the mix is homogeneous. All concrete should be discharged while the mixer drum or blades are rotating.

The required mixing time varies with characteristics of the concrete mix and mixer. A timer should be used to ensure the same mixing time for identical sized batches. Pan-type mixers designed for horizontal countercurrent forced mixing are frequently better for very low slump concrete (0 to 1 in.).

5.3.4.2 Cold weather - When heated water or aggregates are used to warm the mix and satisfy cold weather requirements, the addition of cement should be delayed until after the aggregates and water have entered the mixer and have been thoroughly mixed for at least one minute. This allows the water or aggregates to cool sufficiently to avoid flash set when the cement is placed in the mixer.

5.3.4.3 Hot weather - If the aggregates and water, when combined, have a temperature of over 100 F, the ingredients of the mix should be cooled before mixing to avoid flash set, cold joints or loss of slump. Flake ice or well crushed ice of a size that will melt completely during mixing should be substituted for all or part of the mixing water. To provide products that are uniform in color, it is important that the concrete temperature be controlled between a maximum (90 F) and minimum (50 F) throughout all seasons of the year.

5.3.4.4 Lightweight concrete - Most lightweight aggregates should be pretreated before introduction into the mixer. This will eliminate a loss in workability due to rapid absorption of the mixing water. The lightweight aggregate (and sand, if used) and most of the water are normally placed in the mixer and mixed thoroughly before the cement and remainder of water are added.

5.3.4.5 Architectural concrete - Architectural concrete requires careful observance of good, uniform mixing practices. Mixers must be properly cleaned after each period of production. Blades or liners should be adjusted or replaced per manufacturer’s recommendations to ensure that the mixing is sufficient and adequate. When facing mixes of pigmented concrete with buff or white cement are used in conjunction with gray cement backup concrete, separate mixers and handling arrangements are required. Alternatively when separate mixers are not available, the equipment must be flushed several times and completely cleaned to remove all concrete residue before being used for mixing where specialty cements or pigments are required.

5.4 Reinforcement

5.4.1 Prestressing

5.4.1.1 General - Panels are sometimes prestressed to avoid cracks, to control warping or bowing, or to reinforce particularly large units. Firm anchorage of the prestressing steel in a prestressing bed or in suitably designed individual molds is necessary. The concrete compressive strength when the prestressing force is
Welded as the high temperature may produce crystallization and cause the steel to lose a considerable amount of strength and fail when under tension. Accurate location of strands is important to avoid inducing permanent bowing or warping. Strand ends must be recessed and backfilled with epoxy or special grout, or otherwise carefully protected to avoid corrosion.

Strands are normally tensioned in two increments. The first increment applies enough load to the strands to straighten them, eliminate slack, and provide a starting point for measuring the elongation. The second stress increment is then applied until the strands reach their final stress and elongation. Gauge readings and elongation measurements must be taken and recorded for each strand being stressed.

Prestressing strand, rod, or wires should never be welded as the high temperature may produce crystallization and cause the steel to lose a considerable amount of strength and fail when under tension.

5.4.1.2 Stringing the strands - Strands are normally supplied in reel-less packs where the strands may be pulled from the center of the spool. They should be placed in the form in a way that avoids entanglement during the stressing operation. Strands for pretensioned products should also be free of dirt, oil, grease or any foreign substance which can affect their bond to concrete. The strand chucks should be clean, well lubricated and free from cracks, capable of anchoring the loads induced by the strand without allowing excessive slippage.

5.4.1.3 Jacking - Hydraulic jacks with gauging systems are normally used to tension strands. The hydraulic gauges should be accurate to within 1 percent the applied pressure. Strand force should be determined by observation of jack gauge pressure and measurement of the strand elongation. The two control measurements should agree with their computed theoretical values within a tolerance of ± 5 percent. Additionally, these two values should algebraically be within 5 percent of each other. If the strand readings are not within this range, tensioning should be stopped and corrective measures taken.

5.4.1.4 Strand detensioning - Detensioning should not begin until the concrete has attained adequate strength to resist the compressive forces induced by the strand and to adequately bond the strand to the concrete in pretensioned construction. Strand transfer is normally undertaken when the concrete is at about 3500 psi, but this can vary with design. Each of the strands should be simultaneously and slowly cut at each end of the panel or prestressing bed. Detensioning should be performed in such a manner as to keep forces symmetrical about the panel vertical and horizontal axes.

5.4.2 Reinforcement cage assemblies - Information on detailing and placing reinforcing bars and welded wire fabric may be found in ACI 318, ACI 315, and in the Concrete Reinforcing Steel Institute publications Manual of Standard Practice and Placing Reinforcing Bars. Reinforcement for precast wall panels is usually preassembled into rigid cages using a template or jig before the steel is placed in the form. Cage assemblies should be constructed to close tolerances, and the various pieces should be rigidly connected by tying or welding. When cages are tied, soft stainless steel #16 gauge or heavier tie wire is desirable. Reinforcement cages should be securely suspended from the back of the molds and held clear of any exposed surface. The suspension system must firmly hold the assembly in its proper position during concrete placing and consolidation. Permanent spacers or chairs supported on the form of an exposed concrete surface may mar the appearance of the precast panel. If possible, the use of chairs should be avoided. If used, spacers should be of a type and material that will not cause spalling of the concrete, rust marks, or other deleterious effects.

All reinforcement, at the time concrete is placed, should be free of grease, oil, wax, dirt, paint, loose rust or mill scale, or other contaminants that may reduce bond between steel and concrete or stain the surface of the concrete. Reinforcing steel should not be bent after being embedded in plastic concrete.

5.4.2.1 Welding of reinforcement - Tack welding is only recommended for increased rigidity and should not be indiscriminately used. When absolutely necessary, welding should be done by certified welders with written approval of the engineer-architect and in accordance with the provisions of the American Welding Society (AWS) D1.4.

If coated bars are to be welded, the coating should be removed by acid etching and rinsing the bar in clear water, or by mechanical means such as wire brushing, abrasive blasting, or grinding. All surfaces to be welded should be bright and clean. The clean area should be at least 1 in. larger than the weld area on all sides of the weld.

Welding of galvanized reinforcement requires a special type of welding rod. Welding removes a portion of the zinc coating in the area of the weld. Since zinc fumes are toxic, adequate ventilation must be provided to remove fumes.

Small tack welds give substantial stability to reinforcing bar cages but are not recommended for attachment to main reinforcement due to the possibility of steel crystallization (embrittlement or metallurgical notch). Tack welds which do not become a part of permanent welds of reinforcing steel are prohibited by AWS D1.4, unless approved by the engineer. However, tack welding of reinforcement at locations where neither bar has a structural function should be allowed. For example, welding the ends of the outside bars (within 10 bar diameters from the free end of the bar) may be an aid in fabrication of reinforcing cages. Reinforcing bars should not be welded within 2 bar diameters or a minimum of 2 in., with 3 in. preferred, from a cold bend as this can result in unpredictable behavior of the reinforcing bar at
the bend. Tack welding must be carried out without significantly diminishing the effective steel area or the reinforcing bar area should be one-third larger than required. A low heat setting should be used to reduce the undercutting of the effective steel area of the reinforcing bar.

Welding of higher strength steels should be discouraged unless proper welding procedures, identifying type of preheat and welding rods, are written. Welding should be prohibited near prestressing steel.

5.5-Concrete placement

5.5.1 Transportation - Concrete for casting precast wall panels is transported from the mixer and placed in forms by various methods depending on the precasting operation layout or the type of panel being manufactured. Many precasting plants have stationary mixers and deliver the concrete to the forms by buggies, buckets, conveyors, pumps or other equipment. Some precasting plants operate from a controlled ready-mixed concrete plant and transport the concrete by mixer trucks. Truck mixer delivery (ASTM C 94) is more often used for large volumes of concrete. In addition to speed and economy, avoiding segregation is a prime concern in transporting concrete. Before the delivery of a new batch of concrete, hardened concrete and foreign matter should be removed from the surfaces of the transportation equipment.

5.5.2 Segregation - The amount of segregation varies with the mix consistency and the aggregate grading. Secondary factors which may affect segregation are weather, as it affects consistency, and the mechanism of transportation. Equipment should be used that will provide the least amount of jarring and segregation from the time the concrete is placed in the transport carrier until it is delivered to the forms. Concrete must be discharged into the forms while in its original mixed or plastic state without separation of coarse aggregate and paste.

5.5.3 Consolidation - Concrete used in the manufacture of wall panels should be completely and uniformly consolidated by internal or external vibration, by vibrating screed, by impact, or by a combination of these methods. Although the available consolidation systems vary widely, most have been successful when properly applied. ACI 309 has presented detailed recommendations for good consolidation practice and the control of surface defects.

Even concretes containing high-range water-reducers should be consolidated by minimal vibration. Regardless of the type of consolidation, the goal is to avoid segregation and excessive bleeding while consolidating the concrete into a dense, uniform mass with a surface as free of defects as possible. Four typical vibration techniques are described hereafter. It is recommended that the consolidation method be left to the panel manufacturer.

5.5.3.1 External vibration - External vibration is usually achieved by mounting high frequency vibrators directly on the forms or by using a vibrating table. These vibrators operate at varying frequencies and amplitudes. Vibrating tables or forms should be sufficiently rigid to transmit the vibration uniformly over the entire surface of the panel without any form damage. A vibrating table works best for flat or low-profile units.

5.5.3.2 Drop-table vibration - Drop-table vibration is used in some precasting plants to consolidate concrete with a low total water content. The drop table rises and falls an average of 3/8 in. at a low frequency of approximately 260 cycles per minute.

5.5.3.3 Internal vibration - Internal vibration is done with a tamping type motorized jitterbug or with a spud vibrator. Spud vibrators should not be used to consolidate facing mixes. Because backup mixes are generally thicker and stiffer, they can be placed and vibrated in the same way as regular structural concrete using internal vibration. Vibrators should not be allowed to contact interior form surfaces, because contact may damage the form or mar the concrete surface.

At times, a combination of external and internal vibration is required to properly consolidate the concrete. When high slump concrete is placed, segregation may occur. With normal weight concrete materials, the coarse aggregate tends to settle to the bottom and the fines will rise to the top. With lightweight aggregates, the opposite occurs.

5.5.3.4 Layering - Layering is the fourth consolidation procedure, based on placing differing slumps and mix proportions at different depths in the concrete. This process for consolidation is normally used for deep sections, such as large returns at panel edges. A high cement content and high slump concrete is placed in the first layer, and then in subsequent layers the slump is reduced and the mix changed to provide less fines. Increasing stiffness of the mix in the succeeding layers allows for absorption of excess water and for accommodation of paste from the previous layer. The color of concrete may change as the mix design and slump change. This may be satisfactory if the return is not exposed or the color does not affect appearance.

5.5.4 Facing concrete - Facing concrete should be carefully placed and worked into all parts of the form. This is particularly important in external and internal corners for true and sharp casting lines. Each batch of concrete should be placed as close as possible to its final position. The whole mass should be consolidated by vibration with as little lateral movement as possible. The thickness of a face mix after consolidation should be at least 1 in. or 1.5 times the maximum size of aggregate, whichever is larger. The facing concrete should be thick enough to prevent any backup concrete from showing on the exposed face.

In deep returns, excessive air pockets are often created on the formed surface. This can generally be overcome by rodding the concrete at the return surface with thin round nose sticks after the concrete has been vibrated internally. The technique of two-stage casting or sequential casting is discussed in the next section.
5.6-Surface finishes

5.6.1 General methods - When a precast panel project reaches the fabrication stage, approval of both color and texture will usually have been given. This is generally accomplished by submitting a small sample, or in some cases a full size unit, for approval of the engineer-architect.

Surface finishes can be achieved in many ways, depending on the desired architectural effect. Some surface treatments or finishes are executed on plastic concrete, some on hardened concrete.

Finishes on plastic concrete generally use one of the following methods:

- Chemical surface retarders
- Brooming, floating or troweling of the back face
- Water washing and/or brushing
- Special form finish
- Sand casting
- Surface texturing using formliners
- Clay product veneer-faces
- Stone veneer-faces

Surface treatment of hardened concrete requires more labor and can at times be more susceptible to variations. Available methods include:

- Hand brushing and/or power rotary brushes
- Acid etching
- Sand or other abrasive blasting
- Honing and polishing
- Bushhammering or other mechanical tooling
- Artificially created broken rib texture (hammered ribs or fractured fins)

Additional finishes and further discussion of finish treatments can be found in PCI Architectural Precast Concrete.

Regardless of the type of finishing method, factors such as type and brand of portland cement, aggregates, compressive strengths (at time of final architectural finishing), and curing techniques used will affect the final appearance. When finishes remove part of the surface of the concrete, the resulting panel must have adequate cover over the reinforcement to prevent corrosion and staining.

All methods of finishing must be studied for a project before entering into full production. The precast manufacturer must develop quality requirements for all architectural finishes before undertaking production of such finishes. The finishing process must produce an acceptable uniform appearance without loss of required concrete qualities. When two or more different mixes or finishes are on the same panel, a demarcation (reveal) feature is necessary.

Panels with large or steep returns (such as channel column covers and some spandrels) may be cast in separate pieces in order to achieve matching high quality finishes on all exposed faces as well as minimize air pockets; then they are joined with dry joints as illustrated in Fig. 5.6.1. This method of casting enables all panels to
be cast face-down with the same aggregate orientation and concrete density using conventional precast concrete forming methods; backforming is not required. Also, a combination of face mix and backup mix can be used, rather than 100 percent face mix. If this is the indicated production method, attention must be paid to suitable corner details and reinforcement at the dry joints. Although the dry joint may not show with certain mixes and textures, a groove or quirk will help to mask the joint. Where desired, this joint can be recessed deep enough to allow installation of a small backer rod and placement of a \( \frac{1}{4} \) in. bead of joint sealant, Fig. 5.6.1

5.6.2 Chemical surface retarders - Chemical surface retarders are available in varying concentrations to control the depth of aggregate exposure. They may be used to treat the finished surface whether it is cast up or is on the bottom of the panel, as cast. Retarders being considered for a project should be thoroughly evaluated under prevailing project conditions before production. The retarder selected should be compatible with the particular type and source of cement, aggregates, and specific mix selected for the panels. The effectiveness of surface retarders varies when the heat of hydration of the cement is altered. The heat of hydration may be altered by larger concrete masses, depth of precast product, changes in temperature and/or humidity, by use of insulated panels or by changing cements.

5.6.2.1 Applying the retarder - Chemical surface retarders are specialized chemicals that delay but do not prevent the set of the surface cement paste so that the concrete aggregate can be easily exposed. Form retarders are usually fast drying, solvent-based materials designed to resist the abrasion inherent during the placement of the concrete into forms coated with a retarder. Retarders applied to the top surface of freshly placed concrete are usually water-based materials. Both types of retarders come with retardation strength formulated to produce different depths of reveal. Usually, the one selected after suitable testing will give 30 to 40 percent exposure of the aggregate intended to be exposed at the surface.

Retarders can be applied by roller, brush, or spray. Extreme care should be taken to ensure uniform application of the retarder to the mold or concrete surface.

Since the most suitable period for providing the final surface treatment may vary from 12 to 24 hours after casting, preliminary tests should be performed under job conditions before planning the casting for a large contract. When the concrete matrix is removed by water scrubbing or other mechanical processes, the operation should begin immediately after stripping and before the matrix becomes excessively hard. Unformed surfaces may be treated at anytime after initial set has taken place. Surface retarded concrete is shown in Fig. 5.6.2.1.

5.6.3 Abrasive blasting to expose aggregate - The age of concrete for abrasive blasting is not as critical as for other methods of exposing aggregates. Ideally the concrete should not be more than about 3 to 5 days old, and all panels should have approximately the same compressive strength. The concrete mix used and the compressive strength at time of abrasive blasting affect the final exposure, as do the grading and hardness of the abrasive. Sand or abrasive blasting will produce a muted or frosted

Fig. 5.6.2.1-Aggregate exposed using surface retarders
effect which tends to lighten the color and subdue the luster of the aggregate. The diameter of nozzle, air pressure, and type of abrasive should be determined by experimentation.

If sand is used as the abrasive, the effect of the sand’s color on the panel should be reviewed. With certain combinations of blasting sand grading, pressure, and volume, some of the blasting sand can become embedded in the surface of the concrete. In this case, a blasting sand of similar color to the sand in the concrete matrix should be used. However, this situation can be minimized by a change in the amount of material that hits the surface, the grading of the blasting abrasive and the pressure at the blasting nozzle. Once the blasting sand has been selected, the same sand and grading should be used throughout the project. Surface retarders used in conjunction with sandblasting can help reduce sandblasting time and labor.

The surface of large flat panels should be separated into smaller sections with rustication strips or by the use of ribs and form liners in order to minimize the visual perception of textural differences.

Materials used for blasting operations are washed silica sand, certain hard angular sands, aluminum carbide, blasting grit such as power plant boiler slag, carbonized hydrocarbon, crushed chat, and various organic grits such as ground nut hulls and corncobs. Deep exposure of the coarse aggregate requires a finer gradation of sand abrasive to obtain uniform results. Trials of different abrasive materials with sample panels are made to check the texture and color tones. Sandblasted concrete having light to medium exposure is shown in Fig. 5.6.3.

Exposed aggregate finishes are popular because they are reasonable in cost and provide a good variety in appearance. This variety is achieved by varying the type, color, and size of aggregate, color of matrix, method of exposure, and depth of exposure.

The different degrees of exposure are:

a) Light exposure - where only the surface skin of cement and sand is removed, just sufficiently to expose the edges of the closest coarse aggregate.
b) Medium exposure - where a further removal of cement and sand has caused the coarse aggregate to visually appear approximately equal in area to the matrix.
c) Deep exposure - where cement and sand have been removed from the surface so that the coarse aggregate becomes the major surface feature.

The extent to which aggregates are exposed or “revealed” is largely determined by their size. Reveal should not be greater than one-third the average diameter of the coarse aggregate particles or one-half the diameter of the smallest sized coarse aggregate.

All sandblast operators should be protected with heavy gloves, aprons or protective clothing and air-fed respiratory protection equipment. Caution, relative to environmental pollution, is advised in selecting abrasive grits that produce very fine particles of grit after impacting the surface. Different equipment may be required for wet blasting.

5.6.4 Honing and polishing- Honing and polishing of surfaces provides a smooth exposed aggregate surface.
Honing is generally accomplished by using grinding tools in stages, with successive degrees of grit fineness varying from approximately a No. 24 coarse grit to a fine grit of about No. 300. Polishing can be done with finer grits. Generally, honing alone provides a sufficiently smooth surface for precast panels.

Grinding elements are made with carborundum bonded by resin, or with diamonds set in the grinding or polishing operations are started. Uniform appearance gained adequate strength. Strength of the concrete and of acids is in order. Before acid etching a thorough study is required that a minimum of concrete matrix area show, fill past material should be 5000 psi before any grinding or polishing operations are started. Uniform appearance requires that a minimum of concrete matrix area show, as the aggregates polish better than the concrete matrix.

5.6.5 Acid etching — Due to the many types of aggregates used in architectural concrete, caution on the use of acids is in order. Before acid etching a thorough study should be made of the effect of various concentrations of acid used for exposing aggregates or for the cleaning of panels. The concrete aggregates should be quartz, granite, or other acid resistant stone. Limestones, dolomites, and marbles will either dissolve or discolor when exposed to muriatic acid. Acids may increase chemical reaction between silicates in the aggregate and the free lime liberated from the cement. This may lead to calcium silicate deposits on the panel surface if residue is allowed to harden on surface. Acid washes may also damage the galvanizing of exposed hardware and reinforcing bars if less than recommended cover is used.

When acid is applied, it must be continuously brushed or scrubbed to ensure uniform reaction with the cement surface. Acid washing should not be done until the concrete in the precast panel has reached a minimum strength of 3500 or 4000 psi. All personnel exposed to any acid from the surface application method must wear protective clothing and covering to prevent injury from acid spattering. Acid should be completely neutralized and flushed from the concrete with clear, clean water to prevent yellowing or other discoloration. Acid should not be allowed to remain on the concrete surface for more than 10 to 15 minutes as an absolute maximum. Deep etch exposure should be achieved by multiple treatments rather than prolonged contact.

5.7-Concrete curing

5.7.1 Introduction — Curing of concrete takes place as long as sufficient moisture is present and favorable temperatures are maintained. In the manufacture of precast concrete, the initial curing usually takes place in the form. Secondary curing takes place after the product is removed from the form. Secondary curing may be less important in precast concrete because design strengths are established to enable the panels to resist maximum stresses usually occurring during stripping and handling. Concrete mixes for precast panels generally contain Type III highearly-strength cement or very finely ground Type I cement, with cement contents high enough to assure adequate strength at stripping (usually at least 3000 or 3500 psi). This strength often is achieved within 12 to 16 hours while the precast panel is still in the form. Additional information on curing may be found in ACI 308 and ACI 517.2R.

5.7.2 Curing recommendations — It is recommended that two different stages of curing be established for precast panels. The first 16 to 20 hours is the initial stage and the most crucial. Steps should be taken during this period to both provide heat (if necessary to maintain minimum temperatures) and to prevent loss of moisture from the panel. The exposed portion of the fresh concrete in a wall panel should be covered during this initial phase.

After removal from the form, the secondary stage of curing should be continued until a compressive strength of 3500 psi has been attained and confirmed by standard tests. During this period, precast panels should be protected from excessive moisture evaporation and from temperatures below 50 F.

It may be necessary to interrupt the secondary curing to examine the surface finish and to do any required patching. It cannot be overemphasized that curing at the early ages is extremely important to the strength and durability of the concrete panel.

5.7.3 Curing techniques — Any changes in curing techniques during a given production run may result in changes in color, texture, or uniformity of the wall panels. Therefore, curing procedures should be consistent and uniform from precast panel to precast panel as well as from day to day. Burlap and other similar coverings may cause staining or discoloration on certain finishes and should be avoided in these cases. Because of their tendency to discolor, curing compounds should not be used, except on the backs of panels before the removal of forms, or on surfaces that will receive a finish later. Since some curing compounds and sealers may interfere with adhesion of surface coverings, coatings, and joint sealants, compatibility with these materials should be investigated.

5.7.3.1 Curing temperature — All curing of concrete should take place at temperatures above 50 F. If temperatures fall below this level during the first 20 hours, either external applied heat or heat retention measures are required. However, it must be remembered that trial mix proportions are usually prepared and tested at room temperatures of 70 to 75 F. When concrete is cured at lower temperatures, such as 50 F, the concrete strength may be lower than laboratory tests have indicated. If panel curing temperatures are expected to be lower than trial mix temperatures, some allowance should be made
for the slower gain in strength and the early form stripping time adjusted accordingly.

5.7.3.2 Steam curing — Curing with steam simultaneously provides both heat and moisture. Where steam curing is used, recommended procedures such as those of ACI 517 must be observed to achieve desired results. Steam should not be applied until after the initial set period of the particular concrete mix. The initial set (delay) period should be determined by ASTM C 403. The rate of temperature rise can be from 40 to 80 degrees F per hour as long as a proper initial set (delay) period precedes the heating period. Maximum temperature should not exceed 180 F. Cooling rates should also be carefully controlled. Extremely close control of steam curing procedures is required in connection with chemically retarded exposed aggregate surfaces. Steam curing can produce a greenhouse effect (dripping of moisture from the covering on to the panel) and induce staining on the exposed panel surfaces.

5.7.3.3 Curing in storage — Strength gain can continue after panels are moved to the storage area. Care should be taken to prevent rapid loss of moisture when panels are placed in a storage yard. When ambient relative humidity is high, additional protection from rapid drying may not be required. In areas where hot, dry weather prevails, care should be taken to allow the panels to dry slowly.

New concrete is vulnerable to damage from even one freeze-thaw cycle until it reaches a nominal compressive strength of about 500 psi, since the concrete is in a saturated condition at early ages. At a typical concrete strength above 3000 psi, early freezing is not a problem and the precast concrete panels may be immediately stored outside. However, panels of any strength will not be durable to repeated freezing and thawing unless adequate air entrainment is provided. Section 5.3.3 recommends that all concrete mixes should be air-entrained.

5.8-Storage

5.8.1 General — Because of the wide variation in precast panel sizes and shapes as well as in the production facilities, there are no “standard” methods of handling and storage. Precast panels temporarily stored in a general storage area should be supported at the blocking points designated on the erection drawings. Units should be stored in a vertical or near vertical position. Handling and storage procedures selected should not cause structural damage, detrimental cracking, architectural impairment or permanent distortion when the precast member is being: (a) lifted or stripped from the mold; (b) moved to various locations for further processing or storage; (c) turned into various positions to provide access for finishing and/or surfacing operations; (d) stored before delivery; and (e) loaded onto delivery vehicles.

5.8.2 Protection — During storage, the manufacturer should keep the precast concrete panels in a clean, properly protected area to prevent staining. This does not mean the panels need to be a covered area or must be covered. The need for protection will depend on the configuration of the units, the length of storage time, and the local environment. To protect against freezing damage, inserts and other embedded items should be protected against penetration of water or snow during cold weather.

Storage must be planned carefully to ensure delivery and erection of the panels in an acceptable condition. Even though the panels may require washing after erection, protection may still be necessary against engine exhaust fumes or soil staining.

5.8.3 Storage of thin flat panels — Flat panels less than 4 in. thick or panels with a length to thickness ratio greater than 60 should be stored and shipped in a vertical or near vertical position. Two-point supports spaced approximately at the fifth points are recommended for storage or lifting. Protective resilient material should be provided at points of bearing and contact. All blocking, packing, and protective material should be clean and of a type that will not cause damage, staining, or disfigurement of the precast panels. Blocking and support members should be positioned and secured in a manner to prevent slippage, chipping at the chains, excessive binding, or excessive stresses. Staggered or irregular blocking should be avoided. Precast units should be stacked so they are supported on both sides to equalize loading and to avoid overturning.

5.9-Delivery

Most precast panels are delivered over the highways by semitrailer trucks (see Fig. 5.9). A few are shipped by rail, barge, or other modes of transportation. Precast plant facilities do not generally restrict the size and weight of precast panels which can be produced. However, shipping problems with oversize panels may greatly increase the cost of construction or delay completion of the project. Special permits are generally required where height, width or weight exceed specified limits. The use of lightweight aggregate concrete panels can, in many cases, minimize the impact of weight in shipping, handling and erection operations. Travel may be restricted to good weather, daylight hours and weekdays. Equipment, such as lowboys or special trailers, may be required for large or heavy panels.

All precast panels should be delivered to the site clearly marked as indicated on the erection drawings, with the date of production and an identifier that shows the final position of the unit on the structure. Precast panels should be selected from storage, loaded, and delivered in the proper order to meet the predetermined erection sequence.

Before scheduling of delivery equipment, a field check of the project should be made by the erector to ensure that the foundations, walls and structure generally are suitably constructed to receive precast panel units. The site should be checked for crane and delivery truck access, as well as possible field panel storage. Most panels
Fig. 5.9-Delivery/transportation of panels

should be loaded vertically and supported on A-frames mounted to flat bed trailers. They should be supported to minimize the effect of road shock and should be securely fastened with all contact points protected from damage. Corners and returns of unusual lengths should be braced from edge to edge for greater protection in transit. All material in contact with the panel should be nonstaining.

If panels are shipped horizontal, they should be supported at two points with the supports located at the fifth points of the long dimension to avoid excessive stresses which may be induced by twisting and/or racking of the trailer. When the two-point support system is impractical, alternate support systems must be engineered and checked for feasibility relative to stresses and other potential problems.

5.9.1 Protection during shipping - Generally, protective covering of a precast panel during delivery should be determined by the manufacturer after considering such factors as size, shape, type of finish, type of aggregate, the method of transportation, type of vehicle, weather and road conditions, and distance of haul. Since manufacturers are responsible for the condition of the delivered product, they make the decision on wrapping unless the engineer-architect has specified a particular form of wrapping protection.

5.9.2 Economical panel sizes - Economical panel sizes depend on the plant capability, distance to the job site, highway conditions, and shipping and erection restrictions. For maximum economy, panels should be limited to a height and width of 8 ft and a weight of 20,000 lb to allow for two on a truck. In some areas, heights of 13 ft 6 in. are allowed without special permit, while in others this limit is 12 ft. Restrictions generally exist for loads over 8 ft in width; maximum permit widths can vary from 10 to 14 ft, depending on area or city. Some areas allow overall lengths over 70 ft with a simple permit, escorts front and rear and travel limited to certain times of day.

CHAPTER 6-INSTALLATION

6.1-Planning and preparation

6.1.1 Coordination - Early in the construction, before panel manufacture, the panel fabricator, erector, engineer-architect, owner, and the general contractor should hold a coordination meeting to establish the working relationship, assure that handling techniques are satisfactory, establish the temporary erection bracing and establish mutually agreeable delivery schedules.

6.1.2 Access - Access conditions at the site should be reviewed, considering temporary roads for delivery trucks and handling equipment such as cranes. Responsibility for sidewalks, overhead lines, barricades, truck space at site, sequences, coordination with other trades, and panel protection should be discussed at a project coordination meeting. At this coordination meeting the precast erector should provide his scheme for the handling, loading, transportation and erection of the panels. Temporary bracing of the structure, on-site storage, connections, starting-location and sequence of erection relative to building stability should be discussed.

6.1.3 Project meetings - During the precast erection phase, the general contractor should conduct frequent project meetings between the erector and those subcontractors whose work is affected by the precast. These meetings will help to ensure that all necessary provisions have been made to facilitate the erection process.

6.1.4 Contract documents - The contract documents should state clearly any requirements or sequencing of
erection needed to maintain building stability. Limitations on loading of the structure, temporary bracing requirements, or elevation sequencing need to be clearly shown before bidding. All details of temporary erection bracing, temporary connections, and shoring should be reviewed by the engineer-architect. The sequence of removal of any temporary erection connections should be shown, since leaving these connections in place can result in structural behavior not intended by the engineer-architect.

6.1.5 Pre-erection check — Before starting erection, bearing walls, foundations, structural frame, bearing surfaces, notches, embedded plates, angles or bolts, and welded connections should be checked for dimension, location, line, and grade to ensure that the area to receive the precast panels is ready. Any modification to bearing surfaces or connection hardware should be made by the general contractor before erection begins. The precast erector should also spot check the access before scheduling the loading and handling equipment.

6.2-Unloading and handling

6.2.1 General — Panels should be loaded and delivered in the erection sequence established at the project coordination meetings. Ideally panels should be lifted from the delivery trucks and placed directly in their proper position on the building. This minimizes handling damage and is usually the most economical method. Many requirements in Chapter 5 concerning the handling of panels at the fabrication plant, apply equally to jobsite activity.

The precast erector should set out joint location and spacing before actual panel installation. This should minimize differential variation in the panel joint width as well as identify problems caused by the building or the adjacent materials being out of dimension or alignment.

6.2.2 Delivery sequence - A delivery sequence for panels should be sufficiently flexible to allow for:

- Full loads, using reasonable “fill-out” units if necessary
- Control of unit position on the trailer with proper support for safety and economy
- Adequate advance notice of shipment
- Assurance of prompt unloading
- Provision for some on-site storage

If possible, panels should be unloaded by handling in a vertical position. This is usually the situation if single story panels are shipped on frames in a vertical or an upright position. All chains, binders, banding, protective packing and bracing should be carefully removed from around the panels. Corners and panels with returns of unusual length are shipped with special bracing which should not be removed until the precast piece has been lifted slightly from the truck before installation. If belts are used in unloading, only one panel at a time should be removed. Protective material must be used between the belts and point of contact with the panel. Gangs of precast panels should not be removed with belt lifting devices unless the panels are palletized.

The exterior panel should always be unloaded first from a frame or a stack; never slide a panel out from the middle of a stack. Balance on the trailer should be maintained during unloading by unloading alternate sides of the vehicle. Remaining adjacent panels on the trailer should be tied or blocked to prevent tipping.

After delivery, a panel may require rotation into a new position; for example a tall panel delivered on its side must be rotated to a vertical position. A panel may also be delivered flat (horizontal) and then be lifted from the delivery vehicle and uprighted in the air. The panel is normally rotated without being allowed to touch the ground. It may be necessary to bolt a support frame to the panel before rotating. Usually two lifting lines from the crane are used, although special rotating frames have been developed for use with one crane line.

6.2.3 Lifting devices — Lifting devices should be secured to panels in accordance with the lifting device manufacturer’s recommendations. Bolts must be threaded into the inserts 2.5 bolt diameters for coil inserts and 1.5 bolt diameters for ferrule inserts to prevent stripping of the bolt or insert threads. At least two connections should be used whenever the panel is lifted, so that the panel or the lifting line cannot spin and unscrew, causing the lifting line to become disconnected. Bolts of proper length must be used to ensure a full embedment in the lifting device. Regardless of the load requirements, a 1/2-in. bolt should be the minimum size used for any precast panel handling. Expansion bolts, predrilled or self-drilled, should not be used for handling and erection purposes. Occasionally inserts, bolts, or other devices are provided only for the convenience of field handling. When these devices are located in finished edges or exposed surfaces, bolt and insert holes require filling and repairing. When this is necessary, the engineer-architect should be advised so that the locations and repair procedure can be approved before panel fabrication. Repairs should be properly executed in accordance with Section 6.6.

6.3-Jobsite storage

6.3.1 General — If the jobsite storage of precast panels is necessary to meet the established schedule, the storage areas provided should be relatively level, firm, well-drained and located where there is little chance of damage due to other construction activity. Recommendations for storage in Section 5.8 should be followed. In addition, where long-term storage is necessary, panels should be covered to protect them from accumulation of dust, dirt, or other staining materials. Covers of canvas, rubberized sheets, heavy waterproof paper, reinforced plastic sheeting, or other protective material should be considered.

The storage area may have to be stabilized so that differential settlement or twisting of the stored panel will not occur. Panels should not be stored on frozen ground
without proper safeguards to prevent uneven settlement if the ground thaws.

6.3.2 Panel support - Panels should be stored with identification marks clearly visible and supports at the blocking points shown on the erection drawings. Panels should be blocked to prevent tipping. When panels are placed against a frame or support, they should be set on protective material laid horizontally under or between the panels. This protective material should be selected so that the blocking material will not stain the panels. Plastic chairs, chain guards, and bearing pads are readily available and do not stain. Wood blocking should be wrapped in plastic sheeting to avoid wood stains that can be serious enough to cause panel rejection. Polystyrene foam blocking may dissolve when the solvents of sealers contact it, leaving an unsightly patch of polystyrene that is virtually impossible to remove without defacing the panel. When solvent-based sealers are not used, polystyrene foam can provide good protection if it is of adequate size to support the imposed load.

6.3.3 Storage on a delivery vehicle - If jobsite storage is limited or of short duration, leaving the panels on the delivery truck is often more desirable, provided the shipper will permit extended truck usage. Leaving the panels on the truck provides a clean safe place and eliminates extra handling. This reduces possible damage caused from multiple handling and improper jobsite storage techniques.

6.4-Installation

6.4.1 Workmanship - Workers should be properly trained to handle and erect precast concrete panels. Methods of erection should be planned to avoid soiling, cracking, chipping and damage to built-in items. Chipping and spalling may be repaired at the jobsite after installation, if done to the satisfaction of the engineer-architect.

6.4.2 Equipment - Handling equipment for precast panel erection should be safe and reliable under all anticipated conditions to which it will be exposed. It must not only accomplish the handling/erection quickly and economically but also eliminate any possibility of hazard to personnel on the site, to the public nearby, or to property. Factors involved in equipment selection include:

- Mobility and cost — availability and cost of the handling equipment; cost of altering boom length or making other modifications; mobility needed for anticipated site conditions; whether the panels will be walked or carried
- Capacity required — the weights, dimensions, and lift radius of the heaviest and largest precast panel; the maximum lift height and radius and the weight to be handled at that elevation; the number and frequency of lifts
- Clearance needs — the clearance between the load and adequate headroom in which to operate; ground clearance and conditions of the ground on which to set the equipment; overhead clearance of wires and adjacent buildings.

The equipment selected must meet or exceed all project requirements and have at least a 5 percent working margin of reserve load capacity on every lift for unanticipated problems. When slings are used for panel erection, the included angle between the sling lines should never exceed 90 deg (or 45 deg from the vertical).

Lifting devices must be checked to assure that their capacity and intended use conforms to the manufacturer’s recommendation. Panels should be handled only at the locations and with the hardware shown on the erection shop drawings. If slings are used, the panel should be marked so that the slings are placed in the proper location.

6.4.3 Bracing and guying - Bracing requirements should be established before bidding so that proper allowances can be made. Necessary bracing and guying material should be delivered to the jobsite before erection begins. All bracing and guying methods must be designed to support all construction loads including wind. Building design should provide for structural stability during erection of the precast panels. Until proper alignment and final connections are made, structural stability may not be achieved and bracing may be required.

When bracing/guying is used, the manufacturer’s recommendations must be followed regarding load, length, and inclined angle. Special care must be given to the location, size and capacity of the insert in both the panel and the deadman or floor slab. A brace/guy should never be connected to a precast panel at a point lower than two-thirds of the panel height. Temporary bracing or guying should be arranged so that it does not interfere with other panels being erected, nor should removal of one brace remove support from the remaining panels. Removal of temporary bracing/guying should not take place until the building stability has been achieved through other means or until authorized by the engineer-architect.

6.4.4 Alignment - Offset lines are normally marked on the floors for multistory buildings or on foundations for single-story buildings. Elevations for precast panels are normally established by setting the properly sized shim pack on the floor or beam. Shim material should have a bearing capacity of approximately 1000 psi. Each panel should be erected to meet the tolerances of Chapter 3. To hold overall building dimensions, it is necessary to work to joint center lines, permitting the joint widths to vary. If a joint size is detailed as 1/2 in. and the panel tolerance is ± 1/8 in., the joint may vary from 3/8 to 5/8 in., provided approved connection and erection procedures are followed.

6.4.5 Connections - Connections should be compatible with both precast and supporting frame tolerances, be simple in detail, and easily adjustable in the field to meet special project conditions. Connections should allow
erection to proceed independent of ambient temperature without temporary protection measures. They should be as standardized as possible in order to minimize plant and field erection quality control problems. Once agreed upon by the engineer-architect, the erector, and the precast supplier, the typical connections selected should be shown on the shop drawings submitted before panel fabrication.

6.4.5.1 Bolted connections - Connections should be designed so that members can be safely secured to the structure in a minimum amount of time without completing alignment and all adjustments. Bolted connections are positive immediately and allow for adjustment without tying up large handling equipment. Care must be taken during adjustment to prevent damage to either the panels or the adjacent building materials.

Standardized attachment hardware (clip angles, bolts, and shims) helps to minimize errors and control inventory. Regardless of the load requirements, a 1/2-in. diameter bolt should be the minimum size used. Clip angles should be slotted or have oversized holes to allow for product tolerances and building movement.

6.4.5.2 Welded connections - Where welded connections are required, welding should be done in accordance with AWS D1.1 and with the erection drawings. These drawings should show the type, size, length of weld, sequence, minimum preheat, interpass temperature, weld location and, if critical, the type of electrode. Panels may be shimmed while the initial tack welding is done. Bracing or other provisions must be adequate to safely hold the panel in position while the handling equipment is released and adjacent panels are placed.

Before temporary bracing is released, the designated full weld should be in place at every connection in the precast panel. To minimize staining, all loose slag and debris should be removed immediately after the welding is complete. Panel finish and surrounding materials may require protection from sparks and smoke stain. Non-combustible shields should be used to protect exposed concrete surfaces during welding.

6.4.5.3 Post-tensioned connections - Post-tensioning, either vertical and/or horizontal, may be used for field connection of precast panels using either bonded or unbonded tendons. Bonded tendons are installed in preformed voids or ducts; they are made monolithic with the member and protected from corrosion by grouting after the stressing operation is completed. The grout must fill all voids in and around the tendon for its entire length. Unbonded tendons are connected to the precast panel only through the anchorage hardware. Anchorage devices for all post-tensioning systems must be aligned with the direction of the axis of the tendon at the point of attachment. Concrete surfaces against which the anchorages bear must be in the plane of the tendon and normal to the tendon direction. Post-tensioning operations require personnel properly qualified and experienced with the stressing procedures and equipment to be used.

6.4.5.4 Dowels and grouting - The strength of a dowel connection in tension or shear depends on the embedded length and the developed bond. Since placement of a portland cement grout or epoxy grout is required during erection, use of dowel connections usually slows down the erection and may be costly.

For doweled or grouted connections, setting shims are located and grout holes are filled just before setting precast panels. The concrete in and adjacent to the grout holes should be damp or in a saturated surface dry condition. Grout consistency should permit displacement of some grout when panels are placed in position. Where grout beds are required, the panels may be set on shims and dry-packed with mortar later. Panels may also be set onto fresh grout with the elevation controlled by shims. Excess grout should be removed if it interferes with other construction activities. Use of epoxy and cementitious grouted connections should be avoided when the ambient temperature is below 40 F. Mixing and installation of epoxy grouts must be in strict accordance to the manufacturer’s instructions. In selecting methods of doweling and grouting, consideration must be given to how the final joint will be made and how the corners will be joined. Adjustments to the precast panel after the initial set of the grout may destroy the grout bond and reduce the connection strength. Doweled and grouted connections should only be used where they are part of the structural design concept.

6.5-Cleaning

6.5.1 Protection - Panels should be delivered to the jobsite in a clean and acceptable condition. The erector should recoat all welds and abraded steel with a rust inhibitor or, in cases of galvanized plates, with a cold galvanizing coating. The erector is normally responsible for any chipping, spalling, cracking and other damage to the precast panels after delivery to the jobsite. The general contractor assumes responsibility for panel protection after panel erection.

Any mortar, grout, plaster, stains, or other matter and droppings on the panels during the course of general construction should be immediately washed off with clean water or cleaned as otherwise required. Rainwater or water from construction hoses can wash across building materials and cause discoloration of exposed precast panels.

Arrangements should be made by the general contractor to provide protection for adjacent materials (such as glass and aluminum) which may be damaged by welding. Otherwise the adjacent materials should not be installed until the panels are in place, repaired, and cleaned.

If cleaning is required, exposed panel faces should be washed with a cleansing agent mixed with hot water. Panels should be thoroughly rinsed with clear water after washing. A good fiber brush should be used for cleaning. Normal procedure is to begin cleaning panels from the top of the building downward. Individual panels are cleaned by starting at the precast panel bottom and
working up. After first washing from the bottom to the top, the panels should be rinsed and then washed from top to bottom, followed by a second rinse with clear water. Cleaning solutions must never be allowed to dry on the concrete surface that will be exposed to view. The final finish should be sound and have exposed concrete free of all laitance, dirt, stains, smears, or other blemishes.

6.5.2 Stubborn stains - If stains remain after cleaning with a stiff brush and cleansing agent, the surface of the panel should be thoroughly wetted with clear water and then scrubbed with a dilute solution of muriatic acid. The concentration of this solution may be increased to a maximum of 5 percent muriatic acid, but weaker solutions should be tried first. A thorough washing with clear water should immediately follow the scrubbing. Acid cleaning cannot be done on honed and polished surfaces or if there are soluble aggregates such as limestone or marble on the panel face. Acid cleaning may change the appearance of sandblasted surfaces. Care must be taken to prevent damage to adjacent material corrodbly by the acid. Glass and aluminum trim are especially susceptible during acid scrubbing and washing. Repeated applications of a cleaning acid on the exposed surface of the precast panel may change the color or texture of the panel. The effect on appearance may necessitate extensive washing of all project panels. Other commercial cleaners may be desirable in lieu of acid. Because muriatic (hydrochloric) acid may leave a yellow stain on white concrete, a 3 to 5 percent phosphoric acid solution may be preferable on white or very light concrete panels.

6.5.3 Sandblasting and steam cleaning - High pressure sandblasting and steam cleaning are also common ways to clean panels. An experienced operator should be engaged for sandblasting of precast panels. Sandblasting may dull the aggregate or change the color or texture so that it no longer matches the remainder of the structure. A small area, preferably on the mockup panel, should be tried and approved before proceeding with the work.

6.5.4 Sealers - If panels that have been sealed before shipping to the site require cleaning, it may be necessary to remove the sealer and recoat the panel after cleaning. Surface sealers should never be reapplied until all repairs and cleaning have been completed.

6.6 Patching and repair

6.6.1 General - Minor chipping of precast panels during transportation and handling at the jobsite can be expected. Damaged panels can be repaired after erection, but major spalls or cracks require an engineering evaluation. Repair work requires expert craftsmanship if the end result is to be both structurally sound and pleasing in appearance. Careful planning is required to determine whether the repairs can economically match the existing surface concrete both in color and texture and be structurally sound. In some cases, it may be more feasible to recast the panel. All patching and repairs should be fully cured, cleaned, and dry before installation of sealant in the joints between panels. General guidelines for repair of concrete are found in ACI 546. The repairs should conform to the contract documents and be architecturally satisfactory. Gross variations in color and texture of repairs from the adjacent surfaces may be cause for rejection and a request for replacement of the panel.

6.6.2 Repair considerations - A certain amount of repair is routinely expected on architectural panels. Evaluate the imperfections in the concrete to determine if repairs should be attempted. Repairs can accentuate the flaws, rather than remove them. Slight color variations can be expected. Attempt hand tooling of surface blemishes before undertaking patch repairs. Needle scaling, bushhammering, or other mechanical tooling can be effective in blending in offsets or variations in color and texture.

Use the jobsite mockup or a damaged reject panel to develop a patch mix design and practice repair and texture techniques. In most cases, it is better to complete the preparation of all finishes on the precast panel before beginning any patching or repairs. Recent repair areas can be damaged if repairs are made ahead of the panel finishing. Epoxy-filled crack repairs should be done before sandblasting. Blasting before the epoxy work will cause the crack to have rounded edges, and it will be more difficult to minimize the crack appearance.

Cure all patches and repair areas. These will require better curing and greater weather protection than the original concrete. Proper tools, staging or scaffolds, and safety or protection equipment should be set for all finishing personnel. Patch mix designs and finishing techniques should be documented so that new workers can take over repairs if others leave the project.

6.6.3 Chips and spalls - Chips, spalls, and areas of unsound concrete may be prepared with a hand-held or pneumatic chisel. The repair area should be chipped out to a depth of 1.5 times the maximum size of aggregate to assist in physically holding the patch mix in place. All dust must be brushed or blown from area to be repaired. The entire area to be repaired, as well as adjacent surfaces, should be prewetted before applying a recommended bonding agent. A stiff predesigned patch mix should be applied onto repair area and compacted for maximum density by dry tamping.

Large repair areas will need coarse aggregate in the patch mix or the aggregate must be hand placed and rodded into the patching grout. Strike the repair area level and add any surface texturing while the concrete is still plastic. Begin curing immediately. If the spalled-out piece of concrete is available and fracture surfaces still mate, the easiest repair is to adhere the spalled concrete back into place using an epoxy adhesive. The fractured surface of the panel and spalled piece should both be painted with the epoxy adhesive. Apply enough epoxy so that some epoxy will squeeze out when the mated pieces are clamped together. Use an epoxy with enough viscosity to prevent sagging or running on vertical surfaces. It may be necessary to drill through the spalled piece and
into the precast panel to set pins or bars to increase anchorage.

6.6.4 Crack repair- Panels may crack during transportation and erection. Structural and visual acceptability of cracks should be determined by the engineer-architect and owner (crack acceptability is discussed in Section 2.5.3.1). The repair method for a crack depends on its size, location, and the engineering problems causing the crack. Cracks that are nonworking or have no significant structural problems may be chipped or routed out and repaired in the same way as chips and spalls (Section 6.6.3).

Cracks that are relatively short but that require structural repair may be chipped or routed out to a minimum depth of 1.5 times the aggregate size and filled with a nonsagging epoxy adhesive or stiff mortar. Where the crack is on the exposed finish face of the panel, the epoxy repair preparation may be taken deeper into the panel to allow for application of a surface patch mix to match adjacent surfaces.

6.6.4.1 Epoxy injection- Longer or deeper crack repairs may require epoxy injection using a low viscosity 100 percent solids material. The epoxy color (amber, gray, white or pigmented) should closely match the grout, hydraulic cement or epoxy resin sealant. The repair method for a crack depends on its size, location, and the engineering problems causing the crack. Cracks that are nonworking or have no significant structural problems may be chipped or routed out and repaired in the same way as chips and spalls (Section 6.6.3). To ensure quality control, sealant subcontractors may be as recommended by the manufacturer. Some primers leave an amber stain if brushed along the exposed panel face. Removal of this stain often requires mechanical means and is expensive. Sealers or sealants should not be applied directly over silicone or acrylic waterproof materials. Further information can be found in ACI 504R.

6.7 Sealant installation- Sealant application should be scheduled when the exterior temperature is from 40 to 90 F. Installation temperature may vary depending upon sealant material, humidity, and protection available. In cold weather, the joint and the sealant may be heated depending upon the manufacturer’s recommendations.

It is desirable to use the minimum amount of sealant to make a satisfactory joint. A relatively thin bead is less likely to fail than a thick bead because the thin bead can deform more uniformly. For joints up to 1/2 in. wide, the depth of the sealant should equal the width. For wider joints, the depth should equal half the width, but not more than 1/2 in.

Sealant material should be delivered in the manufacturer’s original sealed containers with labels intact. Recommendations of the manufacturer should always be followed regarding mixing, surface preparation, priming, pot life and installation procedure. Good workmanship by qualified experienced sealant applicators is the most important factor required for satisfactory performance. To ensure quality control, sealant subcontractors may prefer to mix all two-part sealants at their businesses, load into cartridges, and then flash freeze the cartridges before delivery to the jobsite. If done properly, this will eliminate dust, dirt or moisture contamination. The sealant (caulking) gun should have a nozzle of proper size and should provide sufficient pressure to completely fill the joint. Joint filling should be done carefully and completely, thoroughly working the sealant into the joint. The sealant should be smoothed and neatly tooled to eliminate air pockets or voids. Since some solutions used to facilitate tooling, have discolored lightly colored concrete surfaces, tools should be used dry or wetted only with water when working next to these surfaces. The final surface of the tooled sealant should fill the joint, be smooth and free of ridges, wrinkles, sags and air pockets as much as possible. All sealant smears, primers, solvents and other materials used in caulking or sealant work should be removed immediately and entirely from adjacent surfaces as the work progresses. Use the solvent or cleaning agent recommended by the sealant manufacturer, so long as it does not discolor the exposed surface of the precast panel.

Backup fillers used in joints to control the depth of air blowing may be necessary to remove surface contaminants. After cleaning of the joint area, the joint should be wiped with a cloth dampened with an oil-free solvent, or as recommended by the sealant manufacturer. To ensure good adhesion between the sealant and the panel concrete surface, some sealant manufacturers recommend a primer. The sealant and primer combination should be as recommended by the manufacturer. Some primers leave an amber stain if brushed along the exposed panel face. Removal of this stain often requires mechanical means and is expensive. Sealers or sealants should not be applied directly over silicone or acrylic waterproof materials. Further information can be found in ACI 504R.
sealant should be installed in the joint with a minimum compression of 30 percent. If heavy-wall, hollow-core tubing, block, and rod stock are used as fillers, they should be compressed and rolled into the joint in such a way as to avoid linear stretching. Filler rods or blocks should not be twisted or braided during installation.

The sealant joints described above are called one-stage joints. Several typical situations are shown in Fig. 6.7.2.

6.7.3 Two-stage joints — When a two-stage joint (see examples in Fig. 6.7.3) is used, proper venting and draining are absolutely necessary. All horizontal joints should have vent tubes, generally located at the junction of the horizontal and vertical joints. Venting prevents water (either from penetration or from condensation) from collecting between the wall structure and facing panels or within the panel joints, if both the exterior and interior faces of the panels are sealed.

Vent tubes should be 1/4 to 3/8-in. inside diameter polyvinyl chloride or other nonstaining materials. The vent tube should slope down to the exterior face of the panel and must penetrate the joint backup filler so that it allows for free movement of air between the outside environment and the cavity. A minimum of two vent tubes per panel or spacing at 6 to 10 ft on center should be used. Vents should project at least 1/4 in. (6 mm) beyond the sealant exterior face.

When installing a two-stage joint, apply the interior air seal or sealant first. This will minimize the escape of warm moist air from the interior to the joint cavity, thereby condensation.

6.7.4 Fire-resistant joints — When installing a fire-resistant joint system, the fire-resisting blanket should be installed under a minimum of 10 to 15 percent compression.10

CHAPTER 7-QUALITY REQUIREMENTS AND TESTS

7.1-Introduction

7.1.1 General — An effective quality assurance and quality control program in the manufacturing of precast panels benefits the precaster by reducing the cost of repairing or remaking products due to errors in fabrication. Quality control also reduces the chance of structural failure due to reinforcing bars being omitted or mislocated. Precast erectors benefit when panels meet tolerances so they do not have to spend extra time looking for a satisfactory way to erect out-of-tolerance panels. Overall benefits to the owner accrue in receiving a structure that meets the requirements within the agreed upon budget. The main objective of the quality assurance and quality control (QA/QC) program is to provide comprehensive production inspection and testing so that panels will be made within project tolerances and in compliance with the job specifications.

7.1.2 Materials — Project materials should be evaluated as they arrive to assure that they meet the
(a) Two-stage joint—horizontal—using gasket

(b) Two-stage joint—horizontal—using field-molded sealants

(c) Minimum requirement for elementary two-stage joint

(d) Two-stage joint—vertical—using field-molded sealants

(e) Modified two-stage joint with air chamber

Fig. 6.7.3-Two-stage joints
tions. Inspection records should be kept in a concise, clear form and filed for future reference. Most projects require only minimal testing for concrete compressive strength and absorption and will not require overly detailed inspection of materials. However, there are instances where extensive complicated testing, inspection, and control procedures are needed. This type of sophisticated quality control is beyond the scope of this guide.

7.1.3 QA/QC — Quality assurance and quality control (QA/QC) programs can simplify and improve the interrelation among owner, engineer-architect, contractor, and panel fabricator. In such a program, forms, reinforcement, and embedments should be inspected before concrete placement. The face finish of the panel should be inspected after the panel is cured and stripped from the form. The finish surface of the panels should be inspected for compliance with project requirements for color uniformity and texture before shipment. Inspection procedures should be designed so that panel production can proceed at a prescribed pace with minimum delay.

A good quality control program is simple and consistent. Complicated plant procedures and controls lead to uncertainty and confusion. All persons concerned with production and inspection should be fully familiar with specification provisions including the specified tolerances. It is when tolerances are not properly identified and anticipated that they are most likely to cause delay and panel rejection.

7.1.4 Product finish - There will always be some variation in the color, texture, and finish from panel to panel. Acceptable panels should show no obvious surface defects, other than minor color and texture variations, when seen in good typical lighting at a distance of 20 ft.

The final appearance of the product is difficult to evaluate because it is normally subject to the personal interpretation of the engineer-architect or owner. Where there is a great concern about the color, texture, or uniformity of the panel, the owner should require mock-up panels or samples as recommended in Section 1.4. Once the owner or engineer-architect has inspected the samples and selected an acceptable range of finish and texture, the inspector has a tangible standard for reference.

7.2 Unacceptable defects

Cracks can usually be repaired. The precast fabricator should have an opportunity to make repairs before any panels are rejected. The following list gives definitions of unacceptable panel defects that should not remain on the finished panels:

Casting defects - Excessive bug holes (air voids) on the exposed surface; casting lines evident from different placements and poor consolidation; areas of backup concrete bleeding through the facing concrete; foreign material embedded in the panel face; reinforcement shadow lines

Stains - Rust or other stains; blocking stains or acid stains evident on the exposed surface

Irregularities-ragged or irregular edges; visible form joints or irregular surfaces

Nonuniform color and texture - Panels not matching the approved samples for uniformity of color within a panel or from panel to panel; nonuniformity of aggregate color; adjacent flat and return surfaces with greater difference in exposure than the approved samples

Cracks and repairs - Cracks or repairs visible at 30 ft after final installation and finish.

7.3 Structural adequacy

Precast panels should be inspected carefully to assure that they are structurally sound. When the inspector finds cracks, chips, or spalls in a panel and is unsure of their impact, he should refer the problem to the engineer-architect so that the condition may be evaluated. Even though repairs may be structural, all repairs should match the surrounding concrete and meet architectural requirements.

7.4 Prestressing

Prestressing procedures are described in Section 5.4.1. The strand must be kept untangled and free of any material that will affect bond to concrete. Strand chucks should be clean, well lubricated, and free of cracks.

Hydraulic jacks with gauging systems are normally used to tension the strand. Gauge readings and elongation measurements should be taken and recorded for each strand being stressed. The gauges, which should be accurate to within 1 percent of the applied pressure, should be calibrated a minimum of once a year, or whenever there is cause to question the accuracy of the jack load.

7.5 Materials

7.5.1 General - The panel materials should be continuously evaluated to assure that they meet their respective specifications. In some cases, the material evaluation program involves only obtaining and reviewing mill or material supplier reports. However, in other situations, it may include extensive testing.

7.5.2 Reinforcing steel - Reinforcing steel should be checked to see if it is the proper size and grade. If the reinforcing steel is to be used as an anchor for welded embedment assemblies, the carbon equivalent should meet AWS D1.1 requirements.

7.5.3 Welded assemblies - Welded embedment assemblies should be inspected to assure that the assemblies are fabricated properly with the correct size plates and number of anchors. Welding procedures should be monitored to assure that welds have adequate strengths.

7.5.4 Concrete - In many instances, concrete for precast wall panels is batched at the site of panel fabrication because of the use of special cements and aggregates. The control and testing of this concrete is the responsibility of the precaster.

7.5.4.1 Cement - Cement should be evaluated for
its strength producing characteristics. The cement supplier should provide a certified mill test report with each shipment and also data as outlined in ASTM C 150. Mill test reports should be kept for at least 2 years. It is also beneficial to obtain a 10 lb sample from each delivery and store it in an air-tight container for possible future evaluation in case strength or color problems occur.

### 7.5.4.2 Aggregates

Normal weight coarse aggregates and fine aggregates should meet the requirements of ASTM C 33, except for gradation of aggregates used in the face mix. Structural lightweight aggregates should meet the requirements of ASTM C 330. All aggregates should be sampled in accordance with ASTM D 75, tested for grading in accordance with ASTM C 136, and for specific gravity in accordance with ASTM C 128. Aggregate tests should be made for every 200 cu yd of concrete produced, but not less often than every 2 weeks.

### 7.5.4.3 Admixtures

Admixtures should meet one of the following specifications:

- ASTM C 618 for mineral admixtures
- ASTM C 260 for air-entraining admixtures
- ASTM C 494 for chemical admixtures

Chemical admixture materials should be evaluated in the laboratory or in trial batches in the field to assure compatibility with the cement used in the concrete. If the admixtures are to be used in architectural precast concrete, they should be evaluated for their effect on the concrete color and the consistency of color.

### 7.5.4.4 Mixing water

If the water comes from a municipal water system, it may be used in concrete without further testing. If the water comes from an unqualified source, it should be tested for use in concrete by obtaining a chemical analysis and by making 2-in. mortar cubes as described in ASTM C 109. Baseline data should be obtained using a proven water source. If the temperature of plastic concrete which should be tested daily. When air content tests are made, it is usually quite easy to also determine the plastic unit weight of the concrete.

### 7.5.4.5 Pigments

The pigment supplier should certify that pigments or other coloring agents are resistant to lime and other alkalis and conform to ASTM C 979. A simple test can be made by mixing 20 parts of cement with one part of the pigment, using sufficient water to form a buttery paste. The samples should be kept moist and observed for several days. If considerable fading occurs, the pigments are unsuitable. Under these test conditions, it is possible for the test samples to develop efflorescence. The efflorescence should be removed with dilute muriatic (hydrochloric) acid or 5 percent phosphoric acid followed by copious washing with water before the true color can be evaluated. It takes time to test the durability of a color under the influence of light and sometimes a special artificial light can be used to accelerate aging. Pronounced fading of a colored mortar upon exposure to sunlight for 1 month is evidence that the pigment is unsuitable.

### 7.6-Testing plastic concrete

#### 7.6.1 Consistency

The consistency of concrete should be tested at least once per day for each mix used. A significant variation in consistency is a good indicator of variations from batch to batch in the air content, or a change in aggregate moisture content, grading or density. Two of the more common methods for measuring concrete consistency are the slump test and the Vebe consistometer.

#### 7.6.1.1 Slump

The slump test is an easy, fairly quick test which provides reliable information about the consistency of concrete. ASTM C 143 provides detailed instructions on the proper test procedure. A tolerance of ± 1 in. above the maximum specified slump may be allowed for individual batches provided the slump variation does not affect the appearance or other qualities of the concrete beyond the specification limits. Concrete of lower than usual slump may be used provided that it can be properly placed and consolidated.

#### 7.6.1.2 Consistometer

The Vebe consistometer subjects the concrete to a slump test on a vibrating table (see ACI 211.3). The measure of the concrete consistency is the time, in seconds, required to consolidate the slump cone mass into a 93/8 in. diameter mass. Concretes that require a Vebe time of more than 61/2 seconds may be difficult to consolidate properly with internal vibration. Concretes with Vebe times of less than 41/2 seconds have excellent consolidation characteristics.

#### 7.6.2 Air content

The air content of air-entrained concrete should be tested at least once a day for each mix design whenever strength test specimens are made. An air content check should also be made when any of the following changes occurs:

- The slump varies more than ± 1 in.
- Temperature of the concrete varies more than ± 10 F
- Finishing difficulties develop
- Bleeding appears or increases
- Aggregate grading changes significantly
- There is a change in concrete mix design yield

#### 7.6.3 Unit weight

Unit weight tests of backup concrete should be carried out at least once per week for each mix design used regularly, except for lightweight concrete which should be tested daily. When air content tests are made, it is usually quite easy to also determine the plastic unit weight of the concrete.

#### 7.6.4 Temperature

The temperature of plastic concrete should be recorded whenever strength specimens are cast. In addition, concrete temperatures should be recorded at the start of operations each day and at frequent intervals in hot or cold weather. An armored thermometer accurate to ± 2 F should remain in the sample until the reading stabilizes.

### 7.7-Testing hardened concrete
7.7.1 General - Concrete mixes should meet the durability specifications and achieve the compressive strength criteria outlined in ACI 318, Chapter 4. Generally precast concrete develops strength in excess of the requirements for in-place loads. These higher compressive strengths allow earlier stripping for reuse of forms, more satisfactory attainment of architectural finishes, as well as crack resistance during handling and installation.

7.7.2 Durability - Due to the vertical orientation of most panels, critical saturation is seldom reached and freeze-thaw durability has rarely been a major problem. When durability tests are deemed desirable for precast concrete panels, the tests should follow the procedures of ASTM C 666. A minimum allowable durability factor of 70 is recommended. Air entrainment is recommended for panels subject to freeze-thaw conditions, but a specified fixed air content level is not recommended (see Section 4.5.2).

7.7.3 Absorption - A water absorption test of the proposed facing mixes may provide an early indication of weathering or potential staining properties of the concrete. For the concrete strengths normally specified for architectural precast concrete, a reasonable water absorption should not be a problem unless cement-rich or high-slump nonsuperplasticized mixes or both are used.

7.7.3.1 Procedure for absorption tests - Specimens should be tested after the concrete is 28 days old. The specimens should be dried in an oven at a temperature between 180 and 225°F until the loss in weight in 24 hours is less than 0.1 percent. Test samples should be allowed to cool to room temperature, weighed, and then submerged in water to one-half the specimen height. After 24 hours they should be submerged in water until the water is flush with the specimen top. The water should be both cases be maintained between 65 and 75°F. After another 24 hours, the specimens should be removed, the surface water wiped off with a damp cloth, and specimens weighed on a scale which has a resolution of 0.1 gram. The percentage absorption is the difference between wet weight and oven-dry weight, divided by the dry weight and multiplied by 100. This value may be transformed to volume percentage based on the unit weight of the concrete tested. The maximum water absorption for normal weight concrete face mixes should not exceed about 14 percent by volume.

The specimens should be clean and free of parting agent, form release agent or sealer. Care should be taken not to mix any of these agents into the concrete.

7.7.4 Test specimens - Confusion and discrepancies exist in the selection of the size and shape of test specimens. Many precast plants prefer 4 x 8-in. or 6 x 12-in. cylinders. Others prefer 4-in. or 6-in. cubes for evaluating absorption and compressive strength. Generally the smaller specimens-the 4 x 8-in. cylinder and the 4-in cube-are not used when the maximum size of aggregate in the concrete mix is over 1.0 in. Test specimens should be consolidated, cured, and finished similarly to the products they represent.

7.7.5 Molds - Molds for making test specimens should comply with applicable requirements of ASTM C 31 and C 470. Heavy gauge reusable steel molds, rather than single-use molds of paper or lightweight metal or plastic, are preferred. Any molds that become distorted or do not comply with the dimensional requirements of the appropriate ASTM specification should be discarded.

7.7.6 Compressive strength test specimens

7.7.6.1 Test cylinders - Samples for strength tests should be taken on a strictly random basis as specified by ASTM C 172. If choice of times of sampling or the batches of concrete to be sampled are selected on the basis of appearance, convenience, or other possibly biased criteria, statistical concepts lose their validity. No more than one test should be taken from a single batch and water should not be added after the sample is taken.

Four compression specimens should be made daily for each individual concrete mix (whether facing or backup mix), or for each 40 cubic yards of any one mix where the daily consumption exceeds this volume. Two specimens should be used to determine the stripping strength, particularly if the mix is new and its history not well known. However, one specimen may be sufficient as production progresses. For face mixes, the specimens normally required for determining stripping strength may be omitted when the air temperature is higher than 50°F.

Test specimens should be made and stored as near as possible to the location where they will be cured in accordance with ASTM C 31. Consolidation and finishing procedures should closely follow ASTM C 31 and C 192 requirements.

Capping procedures should be as specified in ASTM C 617 except when fast-setting high strength sulfur compounds, specially manufactured for capping, are used. Compression testing may be performed after the recommended cure time for the capping compound.

7.7.6.2 Cube test specimens - Most of the requirements of Section 7.7.7.1 can be applied directly to cube preparation and testing. Some slight deviations will be required in the consolidation of the concrete as a result of different specimen size and shape. When using cubes, it is reasonable to place the concrete in a single 4-in. layer for the 4-in. cube and two 3-in. layers for a 6-in. cube. Tamping or external vibration should be done in accordance with the appropriate ASTM specification. Similar to the restrictions for 4-in. cylinders, internal vibration should not be used to consolidate either size of cube.

Measured compressive strength for cubes is generally higher than that obtained with concrete cylinders made from the same concrete. Cube data correlation should be made to standard 6 x 12 in. test cylinders if cubes are to be consistently used as the quality control specimen. If no correlation is available, it is recommended that 80 percent of the measured cube strength be used as an estimate of the strength of the same concrete when tested using cylinders.
7.7.6.3 Curing considerations - In the manufacture of precast panels, initial curing of the concrete usually takes place in the forms. Test specimens should be cured with and by the same methods as the units they represent up to the time of stripping the product from the form or mold. If the precast panel is to be steam cured, the test specimen mold should be capable of withstanding elevated temperature without significant distortion.

When the precast panels are removed from their forms, the test specimens should also be removed from their molds and placed in a continuous moist condition at 73.4 ± 3 F. This is the secondary stage of curing. An alternate method for secondary curing may provide the best measure of the potential strength of a particular concrete mix. This method calls for approximately two days of continuous moist curing after removal of the specimens from the molds, followed by storage at approximately 50 percent relative humidity until the test sample is tested. Since this is not an ASTM designated procedure, it should be considered only after the engineer-architect evaluates ambient weather conditions for the project area.

7.7.7 Tests of finished panels - When questions arise about adequate strength of a panel or series of panels, the quality of the actual concrete can best be established by core tests. Alternate methods such as rebound hammer, pullout tests, and penetration probe tests have been used along with pulse velocity testing. Depending on the problem, some equipment and procedures may be better than others in determining the strength of the concrete in the panel.

7.7.7.1 Core tests - Acceptance tests for compressive strength in the past have been limited to the taking of cores. Test cores should always be prepared, conditioned, tested and reported in accordance with the requirements of ASTM C 42. The average strength of three representative cores should be at least 85 percent of the specified strength. No single core should be less than 75 percent of that specified. The length-to-diameter ratio of the core sample should be as close to 2:1 as possible. Any core showing evidence of damage before testing should not be tested and replaced with another core sample.

The engineer-architect should select the location for drilling cores where they will least impair the strength of the structure and the exposed surface finish. Cores holes can often be adequately patched without damage to the appearance or integrity of a precast panel. When possible, cores should be drilled so that the core test load is applied in the same direction as the service load. Often top drilling of core samples minimizes or eliminates damage to the panel face. Since almost all architectural precast panels are cast flat, top coring of the panel produces a representative sample of concrete. Cores taken perpendicular to the face of the panel may be up to 15 percent weaker than cores taken parallel to the face of the panel.

Cores should be drilled with a diamond bit to avoid an irregular cross section and damage from drilling. If the core sample must be broken, wooden wedges should be used to minimize the likelihood of damage. Allow an extra 2 in. of core length at the broken end to permit sawing off ends to plane surfaces before capping.

7.7.7.2 Rebound hammer - Impact (rebound) hammer tests should not be used as acceptance tests for precast panels, but they are of value for qualitative comparisons at the plant for the same job. The rebound hammer test, conducted in accordance with ASTM C 805, can be used to locate areas of lower strength concrete or to track day-to-day variations in the production concrete strength.

7.7.7.3 Pullout tests - The pull-out test, as described in ASTM C 900, provides a direct indication of the tensile strength of the in-place concrete. This test involves drilling into hardened concrete and installing an expansion anchor or embedding an anchor disc during casting of the concrete. The expansion anchor or embedded anchor disc is pulled out perpendicular to the concrete surface, bringing with it a truncated cone of concrete. A relationship between pullout force and compressive strength of the concrete can be developed for a particular project by means of laboratory tests. Good data correlation between field and laboratory compression tests is required for this test to give valid results. These tests are typically done on the back of the precast panel where minimum repair is needed.

7.7.7.4 Penetration probe - The penetration probe test, conducted according to ASTM C 803, is relatively nondestructive, reasonably accurate, and economical. However, an established relationship between the compressive strength and probe test results must be developed for each of the different concrete mixes used in the precast panels. Once the background comparison testing on concrete cores and cylinders has been done, the probe test can give reliable results. Since the amount of penetration is inversely proportional to the strength of concrete, a reading of concrete compressive strength is immediately obtained. Each probe leaves only a small hole which can be easily filled with an epoxy patching compound.

7.7.7.5 Pulse velocity - The principle of the ultrasonic test (ASTM C 597) is that the velocity of longitudinal ultrasonic pulses traveling in solid concrete depends on the density and the elastic properties of the concrete. The test is not a substitute for other methods of evaluating compressive strength, but it is a good method for detection of cracks and cavities, for examination of panel damage due to frost, fire, or chemical attack and for assessment of the relative general quality of concrete.

The test requires access to both sides of the panel. Its use for approximating in-place concrete strength by non-destructive means is limited to those cases where concrete is sufficiently strong to allow pulse transmission at velocities greater than 11,500 ft/sec. Correlations have been established between pulse velocity and such proper-
ties of concrete as density, the static modulus of elasticity, and the dynamic modulus of elasticity. The measured pulse velocity is an average velocity if the facing and backup concrete are significantly different.

7.8-Documentation
Record keeping is an essential part of any quality control program. Management should establish, distribute, and update operational procedures for record keeping so that every person employed in the precasting operation knows what documentation is required. The documentation need not be elaborate; it may require only an outline or the completion of a simple data form. But it is an important part of implementing quality control through control of materials, control of concrete mixes, control of production, control of handling, and control of technical services.

Precasters should develop a rational system of analyzing the production test results and keeping records on materials, breakage/damage, and rejection/rework. Data concerning inspection and test results should be recorded and reviewed particularly when evaluating new materials and products. Record keeping must be such that the characteristics of all precast panels can be identified with a specific mark number that may be tied to a certain mold or form on a specific date.

CHAPTER 8-REFERENCES

8.1-Recommended references

American Concrete Institute

117 Standard Specifications for Tolerances for Concrete Construction and Materials

211.1 Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete

211.2 Standard Practice for Selecting Proportions for Structural Lightweight Concrete

211.3 Standard Practice for Selecting Proportions for No-Slump Concrete

212.3R Chemical Admixtures for Concrete

222R Corrosion of Metals in Concrete

224R Control of Cracking in Concrete Structures

224.1R Causes, Evaluation and Repair of Cracks in Concrete Structures

301 Specifications for Structural Concrete for Buildings

304R Recommended Practice for Measuring, Mixing, Transporting and Placing Concrete

304.2R Placing Concrete by Pumping Methods

304.3R High Density Concrete: Measuring, Mixing, Transporting and Placing

308 Standard Practice for Curing Concrete

309R Guide for Consolidation of Concrete

309.2R Identification and Control of Consolidation-Related Surface Defects

315 Details and Detailing of Concrete Reinforcement

318 Building Code Requirements for Reinforced Concrete

318R Commentary for Building Code Requirements for Reinforced Concrete

503R Use of Epoxy Components with Concrete

503.1 Standard Specification for Bonding Hardened Concrete, Steel, Wood, Brick and Other Materials to Hardened Concrete with Multi-Component Epoxy Adhesive

504R Guide to Joint Sealants for Concrete Structures

517.2R Accelerated Curing of Concrete at Atmospheric Pressure-State of the Art

546R Guide for Repairing Concrete Bridge Superstructures

551R Tilt-Up Concrete Structures

American Society for Testing and Materials

A 36 Specification for Structural Steel

A 108 Specification for Steel Bars, Carbon, Cold-Finished, Standard Quality

A 128 Specification for Steel Castings, Austentic Manganese

A 153 Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware

A 185 Specification for Steel Welded Wire, Fabric, Plain, for Concrete Reinforcement

A 276 Specification for Stainless and Heat-Resisting Steel Bars and Shapes

A 307 Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile

A 325 Specification for High-Strength Bolts for Structural Steel Joints

A 327 Specification for Uncoated Seven-Wire Stress-Relieved Steel Strand for Prestressed Concrete

A 421 Specification for Uncoated Stress-Relieved Wire for Prestressed Concrete

A 490 Specification for Heat-Treated, Steel Structural Bolts, 150 ksi (1035 MPa) Tensile Strength

A 496 Specification for Steel Wire, Deformed, for Concrete Reinforcement

A 497 Specification for Welded Deformed Steel Wire Fabric for Concrete Reinforcement

A 516 Specification for Pressure Vessel Plates, Carbon Steel, for Moderate- and Lower-Temperature Service

A 582 Specification for Free-Matching Stainless and Heat-Resisting Steel Bars, Hot-Rolled or Cold-Finished

A 615 Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement

A 616 Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement

A 617 Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement

A 706 Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement

A 722 Specification for Uncoated High-Strength
Steel Bar for Prestressing Concrete
A 767 Specification for Zinc-Coated (Galvanized) Bars for Concrete Reinforcement
A 775 Specification for Epoxy-Coated Reinforcing Steel Bars
A 779 Specification for Steel Strand, Seven Wire Uncoated Compacted, Stress-Relieved for Prestressed Concrete
A 884 Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement
C 31 Test Methods of Making and Curing Concrete Test Specimens in the Field
C 33 Specification for Concrete Aggregates
C 42 Methods of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C 94 Specification for Ready-Mixed Concrete
C 109 Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm Cube Specimens)
C 136 Test Method for Size and Bulk Density of Refractory Brick and Insulating Firebrick
C 143 Test Method or Slump of Portland Cement Concrete
C 150 Specification for Portland Cement
C 172 Method of Sampling Freshly Mixed Concrete
C 185 Test Method for Air Content of Hydraulic Cement Mortar
C 227 Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)
C 260 Specification for Air-Entering Admixtures for Concrete
C 289 Test Method for Potential Reactivity of Aggregates (Chemical Method)
C 330 Specification for Lightweight Aggregates for Structural Concrete
C 331 Specification for Lightweight Aggregates for Concrete Masonry Units
C 403 Specification for Time of Setting Concrete Mixtures by Penetration Resistance
C 470 Specification for Molds for Forming Concrete Test Cylinders Vertically
C 494 Specification for Chemical Admixtures for Concrete
C 597 Test Method for Pulse Velocity Through Concrete
C 617 Standard Practice for Capping Cylindrical Concrete Specimens
C 618 Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete
C 641 Test Method for Staining Materials in Lightweight Concrete Aggregates
C 666 Test Method for Resistance of Concrete to Rapid Freezing and Thawing
C 803 Test Method for Penetration Resistance of Hardened Concrete
C 805 Test Method for Rebound Number of Hardened Concrete
C 900 Test Method for Pullout Strength of Hardened Concrete
C 962 Guide for Use of Elastomeric Joint Sealants
C 979 Specification for Pigments for Integrally Colored Concrete
D 757 Practices for Sampling Aggregates
E 488 Test Method for Strength of Anchors in Concrete and Masonry Elements

American Institute of Steel Construction
Code of Standard Practice: For Steel Building and Bridges

American Welding Society
AWS D1.1 Structural Welding Code — Steel
AWS D1.4 Structural Welding Code — Reinforcing Steel

Concrete Reinforcing Steel Institute
Manual of Standard Practice
Placing Reinforced Bars

Precast/Prestressed Concrete Institute
Manual for Quality Control for Plants and Production of Architectural Prestressed Concrete (MNL-117)
Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products, Third Edition (MNL-116)
Tolerances for Precast and Prestressed Concrete (JR-307)
Design and Typical Details of Connections for Precast and Prestressed Concrete, Second Edition, (MNL-123)
Architectural Precast Concrete, Second Edition, (MNL-122)
PCI Drafting Handbook - Precast and Prestressed Concrete, Second Edition (MNL-119)

Model Building Codes
Uniform Building Code

8.2-Cited references
5. ACI Committee 533, “Symposium on Precast Concrete Wall Panels,” ACI Special Publication SP-11,
American Concrete Institute, Detroit, Michigan, 1965, 143 pp.


## METRIC CONVERSION

### Conversion to International System of Units (SI)

<table>
<thead>
<tr>
<th>To convert from</th>
<th>to</th>
<th>multiply by</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td></td>
<td></td>
</tr>
<tr>
<td>inch (in.)</td>
<td>millimeter (mm)</td>
<td>25.4</td>
</tr>
<tr>
<td>inch (in.)</td>
<td>meter (m)</td>
<td>0.0254</td>
</tr>
<tr>
<td>foot (ft)</td>
<td>meter (m)</td>
<td>0.3048</td>
</tr>
<tr>
<td>yard (yd)</td>
<td>meter (m)</td>
<td>0.9144</td>
</tr>
<tr>
<td>Area</td>
<td></td>
<td></td>
</tr>
<tr>
<td>square foot (sq ft)</td>
<td>square meter (sq m)</td>
<td>0.09290</td>
</tr>
<tr>
<td>square inch (sq in.)</td>
<td>square millimeter (sq mm)</td>
<td>645.2</td>
</tr>
<tr>
<td>square inch (sq in.)</td>
<td>square meter (sq m)</td>
<td>0.0006452</td>
</tr>
<tr>
<td>square yard (sq yd)</td>
<td>square meter (sq m)</td>
<td>0.8361</td>
</tr>
<tr>
<td>Volume</td>
<td></td>
<td></td>
</tr>
<tr>
<td>cubic inch (cu in.)</td>
<td>cubic meter (cu m)</td>
<td>0.00001639</td>
</tr>
<tr>
<td>cubic foot (cu ft)</td>
<td>cubic meter (cu m)</td>
<td>0.02832</td>
</tr>
<tr>
<td>cubic yard (cu yd)</td>
<td>cubic meter (cu m)</td>
<td>0.7646</td>
</tr>
<tr>
<td>gallon (gal) Can. liquid*</td>
<td>liter</td>
<td>4.546</td>
</tr>
<tr>
<td>gallon (gal) U.S. liquid*</td>
<td>liter</td>
<td>3.785</td>
</tr>
<tr>
<td>gallon (gal) U.S. liquid*</td>
<td>cubic meter (cu m)</td>
<td>0.003785</td>
</tr>
<tr>
<td>Force</td>
<td></td>
<td></td>
</tr>
<tr>
<td>kip</td>
<td>kilogram (kgf)</td>
<td>453.6</td>
</tr>
<tr>
<td>kip</td>
<td>newton (N)</td>
<td>4448.0</td>
</tr>
<tr>
<td>pound (lb)</td>
<td>kilogram (kgf)</td>
<td>0.4536</td>
</tr>
<tr>
<td>pound (lb)</td>
<td>newton (N)</td>
<td>4.448</td>
</tr>
<tr>
<td>Pressure or Stress</td>
<td></td>
<td></td>
</tr>
<tr>
<td>kips/square inch (ksi)</td>
<td>magapascal (MPa)†</td>
<td>6.895</td>
</tr>
<tr>
<td>pound/square foot (psf)</td>
<td>kilopascal (kPa)†</td>
<td>0.04788</td>
</tr>
<tr>
<td>pound/square inch (psi)</td>
<td>kilopascal (kPa)†</td>
<td>6.895</td>
</tr>
<tr>
<td>pound/square inch (psi)</td>
<td>megapascal (MPa)†</td>
<td>0.006895</td>
</tr>
<tr>
<td>pound/square foot (psf)</td>
<td>kilogram/square meter (kgf/sq m)</td>
<td>4.882</td>
</tr>
<tr>
<td>Mass</td>
<td></td>
<td></td>
</tr>
<tr>
<td>pound (avdp)</td>
<td>kilogram (kg)</td>
<td>0.4536</td>
</tr>
<tr>
<td>ton (short, 2000 lb)</td>
<td>kilogram (kg)</td>
<td>907.2</td>
</tr>
<tr>
<td>ton (short, 2000 lb)</td>
<td>tonne (t)</td>
<td>0.9072</td>
</tr>
<tr>
<td>grain</td>
<td>kilogram (kg)</td>
<td>0.0006480</td>
</tr>
<tr>
<td>tonne</td>
<td>kilogram (kg)</td>
<td>1000</td>
</tr>
</tbody>
</table>
### Mass (weight) per Length
- **kip/linear foot (klf)**: kilogram/meter (kg/m) 0.001488
- **pound/linear foot (plf)**: kilogram/meter (kg/m) 1.488
- **pound/linear foot plf**: newton/meter (N/m) 14.593

### Mass per volume (density)
- **pound/cubic foot (pcf)**: kilogram/cubic meter (kg/cu m) 16.02
- **pound/cubic yard (pcy)**: kilogram/cubic meter (kg/cu m) 0.5933

### Bending Moment or Torque
- **inch-pound (in.-lb)**: newton-meter 0.1130
- **foot-pound (ft-lb)**: newton meter 1.356
- **foot-kip (ft-k)**: newton-meter 1356

### Temperature
- **degree Fahrenheit (deg F)**: degree Celsius (C) \( t_c = \frac{(t_F - 32)}{1.8} \)
- **degree Fahrenheit (deg F)**: degree Kelvin (K) \( t_k = \frac{(t_F + 459.7)}{1.8} \)

### Other
- **Section modulus (in.\(^3\))**: mm\(^3\) 16.387
- **Moment of inertia (in.\(^4\))**: mm\(^4\) 416.231
- **Coefficient of heat transfer (Btu/ft\(^2\)-h/F)**: W/m\(^2\)/C 5.678
- **Modulus of elasticity (psi)**: MPa 0.006895
- **Thermal conductivity (Btu-in./ft\(^2\)-h/F)**: W/m\(^2\)/C 0.1442
- **Thermal expansion in./in./F**: mm/mm/C 1.800
- **Area/length (in.\(^2\)/ft)**: mm\(^2\)/m 2116.80

---

*One U.S. gallon equals 0.8321 Canadian gallon
†A Pascal equals one newton/square meter

ACI 533R-93 was submitted to letter ballot of the committee and was approved according to Institute balloting procedures.