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CONTENIDO

SPECIFICATIONS FOR STRUCTURAL CONCRETE FOR BUILDINGS (ACI 301-89)*

COLOCACION DEL CONCRETO BAJO TEMPERATURAS EXTREMAS:

a) COLOCACION DEL CONCRETO EN CLIMAS CALUROSOS

b) COLOCACION DEL CONCRETO EN CLIMAS FRIOS

COLOCACION DEL CONCRETO POR MEDIO DE BANDAS TRANSPORTADORAS

PALACIO DE MINERIA 1992

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Specifications for Structural Concrete for Buildings (ACI 301-89)*

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These specifications are a reference standard which the engineer or architect may make applicable to any building project by citing them in the project specifications. He supplements them as needed by designating or specifying individual project requirements.

The document covers materials and proportioning of concrete; reinforcing and prestressing steels; production, placing, and curing of concrete; and formwork design and construction. Methods of treatment of joints and embedded items, repair of surface defects, and finishing of formed surfaces are specified. Separate chapters are devoted to slab construction and finishing, architectural concrete, massive concrete, and materials and methods for constructing post-tensioned concrete. Provisions governing testing, evaluation, and acceptance of concrete as well as for acceptance of the structure are included.

Keywords: **admixtures; aggregates; air entrainment; architectural concrete; buildings; cements; cold weather construction; compressive strength; concrete construction; concrete durability; concrete slabs; concretes; consolidation; conveying; curing; evaluation; exposed aggregate concrete; finishes; floors; formwork (construction); grouting; hot weather construction; inspection; joints (junctions); lightweight aggregate concretes; materials; mix proportioning; mixing; placing; prestressed concrete; prestressing steels; reinforced concrete; reinforcing steels; repairs; retempering; shoring; specifications; subgrades; temperature; tests; tolerances (mechanics); water-cement ratio; welded wire fabric.**

*Adopted as a standard of the American Concrete Institute, July 1989, to replace ACI 301-84 (Revised 1988), in accordance with the Institute's standardization procedures. It was revised by the expedited procedure effective August 1989.

movement of any part of the formwork system during concrete placement will be prevented.

4.2.13—Runways for moving equipment shall be provided with struts or legs, shall be supported directly on the formwork or structural member, and shall not rest on the reinforcing steel.

4.3—Tolerances

4.3.1—Unless otherwise specified by the architect/engineer, formwork shall be constructed so that the concrete surfaces will conform to the tolerance limits listed in Table 4.3.1.

4.3.2—The contractor shall establish and maintain in an undisturbed condition and until final completion and acceptance of the project sufficient control points and bench marks to be used for reference purposes to check tolerances.

4.3.3—Regardless of the tolerances listed in Table 4.3.1, no portion of the building shall extend beyond the legal boundary of the project.

4.3.4—Permissible variations from plump and designated building lines for portions of buildings more than 100 ft above the ground shall be as specified in the contract documents.

4.4—Preparation of form surfaces

4.4.1—All surfaces of forms and embedded materials shall be cleaned of all accumulated mortar or grout from previous concreting and of all other foreign material before concrete is placed.

4.4.2—Unless otherwise specified or accepted, surfaces of forms shall be treated as follows:

4.4.2.1 Before placing the reinforcing steel or the concrete, the surfaces of the forms shall be covered with an acceptable coating material that will effectively prevent absorption of moisture, prevent bond with the concrete, and not stain the concrete surfaces. A field applied form release agent or sealer of acceptable type or factory applied nonabsorptive liner may be used.

4.4.2.2 Excess form coating material shall not stand in

Table 4.3.1—Tolerances for formed surfaces

1. Variation from plumb:	
A. In the lines and surfaces of columns, piers, walls, and in arrises:	
In any 10 ft of length	1/4 in.
Maximum for the entire length	1 in.
B. For exposed corner columns, control-joint grooves, and other conspicuous lines:	
In any 20 ft length	1/4 in.
Maximum for the entire length	1/2 in.
2. Variation from the level or from the grades specified in the contract documents:	
A. In slab soffits, ceilings, beam soffits and in arrises, measured before removal of supporting shores	
In any 10 ft of length	1/4 in.
In any bay or in any 20 ft length	3/8 in.
Maximum for the entire length	3/4 in.
B. In exposed lintels, sills, parapets, horizontal grooves, and other conspicuous lines:	
In any bay or in 20 ft length	1/4 in.
Maximum for the entire length	1/2 in.
3. Variation of the linear building lines from established position in plan and related position of columns, walls, and partitions:	
In any bay	1/2 in.
In any 20 ft of length	1/2 in.
Maximum for the entire length	1 in.
4. Variation in the sizes and location of sleeves, floor openings, and wall openings	± 1/4 in.
5. Variation in cross-sectional dimensions of columns and beams and in the thickness of slabs and walls:	
Minus	1/4 in.
Plus	1/2 in.
6. Footings*	
A. Variations in dimensions in plan:	
Minus	1/2 in.
Plus	2 in.
B. Misplacement or eccentricity: 2 percent of the footing width in the direction of misplacement but not more than	2 in.
C. Thickness:	
Decrease in specified thickness	5 percent
Increase in specified thickness	No limit
7. Variation in steps:	
A. In a flight of stairs:	
Rise	± 1/8 in.
Tread	± 1/4 in.
B. In consecutive steps:	
Rise	± 1/16 in.
Tread	± 1/8 in.

* Tolerances apply to concrete dimensions only, not to positioning of vertical reinforcing steel, dowels, or embedded items

puddles in the forms nor shall such coating come in contact with hardened concrete against which fresh concrete is to be placed.

4.5—Removal of forms

4.5.1—When repair of surface defects or finishing is required at an early age, forms shall be removed as soon as the concrete has hardened sufficiently to resist damage from removal operations.

4.5.2—Top forms on sloping surfaces of concrete shall be removed as soon as the concrete has attained sufficient stiffness to prevent sagging. Any needed repairs or treatment required on such sloping surfaces shall be performed at once and be followed by the specified curing.

4.5.3—Wood forms for wall openings shall be loosened as soon as this can be accomplished without damage to the concrete.

4.5.4—Formwork for columns, walls, sides of beams, and other parts not supporting the weight of the concrete may be removed as soon as the concrete has hardened sufficiently to resist damage from removal operations.

4.5.5—Forms and shoring in the formwork used to support the weight of concrete in beams, slabs, and other structural members shall remain in place until the concrete has reached the minimum strength specified in the contract documents for removal of forms and shoring.

4.5.6—When shores and other vertical supports are so arranged that the non-load-carrying form-facing material may be removed without loosening or disturbing the shores and supports, the facing material may be removed at an earlier age as specified or permitted.

4.6—Reshoring

4.6.1—When reshoring is permitted or required, the operations shall be planned in advance and shall be subject to approval. While reshoring is under way, no live load shall be permitted on the new construction.

4.6.2—In no case during reshoring shall concrete in beam, slab, column or any other structural member be subjected to combined dead and construction loads in excess of the loads permitted by the architect/engineer for the developed concrete strength at the time of reshoring.

Reshores shall be placed as soon as practicable after stripping operations are complete, but in no case later than the end of the working day on which stripping occurs.

Reshores shall be tightened to carry their required loads without overstressing the construction. Reshores shall remain in place until tests representative of the concrete being supported have reached the specified strength, f'_c, or the strength specified in the contract documents for removal of reshores.

4.6.3—Floors supporting shores under newly placed concrete shall have their original supporting shores left in place or shall be reshored. The reshoring system shall have a capacity sufficient to resist the anticipated loads and in all cases shall have a capacity equal to at least one half of the capacity of the shoring system above. The reshores shall be located directly under a shore position above unless other locations are acceptable.

4.6.4—In multistory buildings the reshoring shall extend over a sufficient number of stories to distribute the weight of newly placed concrete, forms, and construction live loads in such a manner that the design superimposed loads of the floors supporting shores are not exceeded.

4.7—Removal strength

When removal of formwork or reshoring is based on the concrete reaching a specified strength, the concrete shall be presumed to have reached this strength when either of the following conditions has been met:

4.7.1—When test cylinders, field cured along with the concrete they represent, have reached the strength specified for removal of formwork or reshoring. Except for the field curing and age at test, the cylinders shall be molded and tested as specified in Chapter 16, Testing.

4.7.2—When the concrete has been cured in accordance with the provisions of Chapter 12 for the same length of time as the age at test of laboratory-cured cylinders which reached the specified strength. The length of time the concrete has been cured in the structure shall be determined by the cumulative number of days or fractions thereof, not necessarily consecutive, during which the temperature of the air in contact with the concrete is above 50 F and the concrete has been damp or thoroughly sealed from evaporation and loss of moisture.

Notes

In the sections of Chapter 4 listed below, specific acceptance is required:

4.1.4 Of shop drawings for formwork, where required.

4.4.2.1 Of form-coating materials.

4.6.1 Of plans for reshoring, where required or permitted.

4.6.2 Of loads to be permitted on structural members during reshoring.

4.6.3 Of locations of reshores other than specified in this section.

CHAPTER 5—REINFORCEMENT

5.1—General

Placing drawings showing all fabrication dimensions and locations for placing reinforcement and bar supports shall be submitted for review and acceptance. Acceptance shall be obtained before fabrication.

5.2—Reinforcement

5.2.1 *Reinforcing bars*—All reinforcing bars shall be deformed except spirals, which may be plain bars. Reinforcing bars shall be the grades required by the contract documents and shall conform to one of the following specifications:

5.2.1.1 ASTM A 615.

5.2.1.2 ASTM A 616 including supplementary requirement S1.

5.2.1.3 ASTM A 617.

5.2.1.4 ASTM A 706.

5.2.2 *Coated reinforcing bars*—When specified, coated reinforcing bars shall be zinc-coated (galvanized) or epoxy-coated. The reinforcing bars to be coated shall conform to Section 5.2.1.

5.2.2.1 Zinc-coated (galvanized) reinforcing bars shall conform to ASTM A 767. Supplementary requirements S1 and S2 shall apply when fabrication after galvanization includes cutting and bending. Supplementary requirement S2 shall apply when fabrication after galvanization includes only bending.

5.2.2.2 Epoxy-coated reinforcing bars shall conform to ASTM A 775.

5.2.2.3 Repair of damaged zinc coating, when required, shall be made with a zinc-rich formulation conforming to ASTM A 767. Repair shall be done in accordance with the material manufacturer's recommendations.

5.2.2.4 Repair of damaged epoxy coating, when required, shall be made with patching material conforming to ASTM A 775. Repair shall be done in accordance with the material manufacturer's recommendations.

5.2.3 *Bar mats*

5.2.3.1 Bar mats shall be of the clipped type conforming to ASTM A 184 and shall be fabricated from reinforcing bars that conform to Section 5.2.1.

5.2.3.2 Bar mats may be fabricated from zinc-coated (galvanized) reinforcing bars. Metal clips shall be zinc-coated (galvanized). Nonmetallic clips may be used. Coating damage at the clipped intersections shall be repaired in accordance with Section 5.2.2.3.

5.2.3.3 Bar mats may be fabricated from epoxy-coated reinforcing bars. Metal clips shall be epoxy-coated. Nonmetallic clips may be used. Coating damage at the clipped intersections shall be repaired in accordance with Section 5.2.2.4.

5.2.4 *Wire*

5.2.4.1 Wire shall be smooth or deformed wire as indicated on the contract documents.

5.2.4.2 Smooth wire shall conform to ASTM A 82.

5.2.4.3 Deformed wire shall conform to ASTM A 496, size D4 and larger.

5.2.5 *Welded wire fabric*

5.2.5.1 Welded wire fabric shall be fabricated from smooth or deformed wire and shall conform to the wire size and wire spacing required or indicated on the contract documents. Welded wire fabric shall conform to one of the following specifications:

5.2.5.2 ASTM A 185, except welded intersections shall be spaced not farther apart than 12 in. in the direction of the principal reinforcement.

5.2.5.3 ASTM A 497, except welded intersections shall be spaced not farther apart than 16 in. in the direction of the principal reinforcement.

5.2.6 *Spirals*—Spirals may be fabricated from reinforcing bars or wire.

5.3—Wire bar supports

Unless otherwise specified or permitted, wire bar supports shall be in accordance with Class 1, maximum protection, or Class 2, moderate protection in Chapter 3 of *Manual of Standard Practice* by the Concrete Reinforcing Steel Institute.

5.4—Welding

5.4.1—When required or permitted, all welding of reinforcing bars shall conform to AWS D1.4. Unless otherwise accepted, welding of crossing bars (tack welding) for assembly of reinforcement is prohibited.

5.4.2—Welding of wire to wire, and of wire or welded wire fabric to reinforcing bars or structural steels, shall conform to applicable provisions of AWS D1.4 and supplementary requirements specified by the architect/engineer.

5.4.3—After completion of welding on zinc-coated (galvanized) or epoxy-coated reinforcing bars, coating damage shall be repaired in accordance with Section 5.2.2.3 or 5.2.2.4, respectively. All welds, and all steel splice members when used to splice bars, shall be coated with the same material used for repair of coating damage.

5.5—Fabrication

All reinforcement shall be bent cold unless otherwise permitted.

5.6—Fabricating and placing tolerances

5.6.1—Reinforcing bars shall be fabricated in accordance with the standard fabricating tolerances in Fig. 4 and 5 of ACI 315.

5.6.2—Reinforcement shall be placed to the following tolerances:

	Tolerances, in.
Clear distance	
To formed soffit	−¼
To other formed surfaces	±¼
Minimum spacing between bars	−¼
Clear distance from unformed surface to top reinforcement	
Members 8 in. deep or less	±¼
Members more than 8 in. deep but less than 24 in. deep	−¼, +½
Members 24 in. deep or greater	−¼, +1
Uniform spacing of bars, but the required number of bars shall not be reduced	±2
Uniform spacing of stirrups and ties, but the required number of stirrups and ties shall not be reduced	±1
Longitudinal locations of bends and ends of reinforcement	
General	±2
Discontinuous ends of members	±½
Length of bar laps	−1½
Embedded length	
For bar sizes No. 3 through 11	−1
For bar sizes No. 14 and 18	−2

5.6.3—When it is necessary to move bars to avoid interference with other reinforcement, conduits, or embedded items exceeding the specified placing tolerances, the resulting arrangement of bars shall be subject to acceptance.

5.7—Placing

5.7.1—Minimum concrete cover for reinforcement, except for extremely corrosive atmosphere, other severe exposures, or fire protection, shall be as follows:

	Minimum cover, in.
Concrete deposited against the ground	3
Formed surfaces exposed to weather or in contact with the ground	
For bar sizes No. 6 or larger	2
For bar sizes No. 5 and smaller, and W31 or D31 wire and smaller	1½
Formed surfaces not exposed to weather or not in contact with the ground	
Beams, girders, and columns	1½
Slabs, walls, and joists	
For bar sizes No. 11 or smaller	¾
For bar sizes No. 14 and 18	1½

5.7.1.1 For bundled bars, minimum concrete cover shall be equal to the equivalent diameter of the bundle but need not be greater than 2 in.; except for concrete deposited against and permanently in contact with the ground, minimum cover shall be 3 in. The equivalent diameter of the bundle shall be based on a single bar of a diameter derived from the equivalent total area.

5.7.2—All reinforcement, at the time concrete is placed, shall be free of mud, oil, or other materials that may adversely affect or reduce the bond. Reinforcement with rust, mill scale, or a combination of both shall be considered satisfactory provided the minimum dimensions, weight, and height of deformations of a hand-wire-brushed test specimen are not less than the applicable ASTM specification requirement.

5.7.3—All reinforcement shall be supported and fastened before concrete is placed and shall be secured against displacement within the tolerances permitted in Section 5.6.2.

5.7.3.1 Unless otherwise indicated in the contract documents, reinforcement supported from the ground or mud mat shall rest on precast concrete blocks not less than 4 in.^{2} and having a compressive strength equal to or greater than the specified compressive strength of the concrete being placed. Other means of support may be used if accepted.

5.7.3.2 Reinforcement supported from formwork shall rest on bar supports made of concrete, metal, plastic, or other acceptable materials. Where the concrete surface will be exposed to the weather in the finished structure, the portions of all bar supports within ½ in. of the concrete surface shall be noncorrosive or protected against corrosion.

5.7.3.3 Zinc-coated (galvanized) reinforcing bars supported from formwork shall rest on galvanized wire bar supports coated with dielectric material, or on bar supports made of dielectric material or other acceptable materials. All other reinforcement and embedded steel items in contact with galvanized reinforcing bars, or within a minimum clear distance of 2 in. from galvanized reinforcing bars unless otherwise required or permitted, shall be galvanized.

5.7.3.4 Epoxy-coated reinforcing bars supported from formwork shall rest on coated wire bar supports, or on bar supports made of dielectric material or other acceptable materials. Wire bar supports shall be coated with dielectric material for a minimum distance of 2 in. from the point of contact with the epoxy-coated reinforcing bars. Reinforcing bars used as support bars shall be epoxy-coated. In walls having epoxy-coated reinforcing bars, spreader bars where specified shall be epoxy coated. Proprietary combination bar clips and spreaders used in walls with epoxy-coated reinforcing bars shall be made of corrosion-resistant material or coated with dielectric material.

5.7.3.5 Zinc-coated (galvanized) reinforcing bars shall be fastened with zinc-coated tie wire, or nonmetallic-coated tie wire, or other acceptable materials.

5.7.3.6 Epoxy-coated reinforcing bars shall be fastened with nylon-, epoxy-, or plastic-coated tie wire, or other acceptable materials.

5.7.4—Welded wire fabric for slabs on grade shall extend to within 4 in. of the concrete edge. Welded wire fabric may extend through the contraction joints. Welded wire fabric shall be adequately supported during placing of concrete to assure proper positioning in the slab.

5.7.5—Templates shall be furnished for placement of all column dowels unless otherwise permitted.

5.7.6—All splices shall be as indicated on the contract documents unless otherwise permitted. Mechanical connections for reinforcing bars may be used when accepted. After installation of mechanical connections on zinc-coated (galvanized) or epoxy-coated reinforcing bars, coating damage shall be repaired in accordance with Section 5.2.2.3 or 5.2.2.4, respectively. All parts of mechanical connections used on coated bars, including steel splice sleeves, bolts, and nuts shall be coated with the same material used for repair of coating damage.

5.7.7—Bending or straightening or bars partially embedded in concrete shall not be permitted except when specifically accepted. Bending shall be in accordance with Sections 5.7.7.1 and 5.7.7.2.

5.7.7.1 The minimum inside bend diameters shall conform to the requirements of Table 5.7.7, unless otherwise permitted. In addition, the beginning of the bend shall not be closer to the concrete surface than the minimum diameter of bend. Preheating, if required, shall be in accordance with Section 5.7.7.2. The following requirements shall be adhered to for individual bar sizes:

Bar size	Bend requirements
No. 3 through No. 5	Bars may be cold bent the first time. Cold bend bars only when temperature is above 32 F. Preheating is required for subsequent straightening or bending.
No. 6 and larger	Preheating is required.

Table 5.7.7—Minimum diameter of bend

Bar size	Minimum diameter
No. 3 through 8	6 bar diameters
No. 9, 10, 11	8 bar diameters
No. 14 and 18	10 bar diameters

5.7.7.2 Preheating prior to bending or straightening, when required, shall be in accordance with the following requirements:

(a) Preheating may be applied by any method which does not harm the bar material or cause damage to the concrete.

(b) The preheat shall be applied to a length of bar equal to at least 5 bar diameters each way from the center of the bend except that preheat shall not extend below the surface of the concrete. The temperature of the bar at the concrete interface shall not exceed 500 F.

(c) The preheat temperature shall be 1100 to 1200 F.

(d) The preheat temperature shall be maintained until bending or straightening is complete.

(e) The preheat temperature shall be measured by temperature measurement crayons, contact pyrometer, or other acceptable method.

(f) Heated bars shall not be artificially cooled until the material temperature is less than 600 F.

5.7.7.3 *Repair of damaged coating*—When zinc-coated (galvanized) or epoxy-coated reinforcing bars are field bent, coating damage shall be repaired in accordance with Section 5.2.2.3 or 5.2.2.4, respectively.

5.7.8 *Zinc-coated (galvanized) reinforcing bars*—Coating damage due to handling, shipping, and placing shall be repaired in accordance with Section 5.2.2.3.

5.7.9 *Epoxy-coated reinforcing bars*—Equipment for handling epoxy-coated reinforcing bars shall have protected contact areas. Bundles of coated bars shall be lifted at multiple pickup points to prevent bar-to-bar abrasion from sags in the bundles. Coated bars or bundles of coated bars shall not be dropped or dragged. Coated bars shall be stored on protective cribbing. Fading coating color shall not be cause for rejecting epoxy-coated reinforcing bars. Coating damage due to handling, shipping, and placing need not be repaired in cases where the damaged area is 0.1 in.^{2} or smaller. Damaged areas larger than 0.1 in.^{2} shall be repaired in accordance with Section 5.2.2.4. The maximum amount of damage including repaired and unrepaired areas shall not exceed 2 percent of the surface area of each bar.

Notes

In the following sections of Chapter 5, specific acceptance is required:

5.1.1 Of placing drawings for reinforcement and bar supports.

5.4.1 For welding not indicated on the contract documents and to weld crossing bars for assembly of reinforcement.

5.6.3 For relocating bars to an extent that causes placement tolerances to be violated.

5.7.3 Of bar supports and material for fastening zinc-coated (galvanized) and epoxy-coated reinforcing bars.

5.7.5 To allow placement of column dowels without using templates.

5.7.6 For splices not indicated on the contract documents and for the use of mechanical connections.

5.7.7 To field bend reinforcing bars partially embedded in concrete.

CHAPTER 6—JOINTS AND EMBEDDED ITEMS

6.1—Construction joints

6.1.1—Joints not indicated on the contract documents shall be located and constructed to minimize the impact on the strength of the structure. Joint types and locations shall be acceptable to the architect/engineer. In general, joints shall be located near the middle of the spans of slabs, beams, and girders unless a beam intersects a girder at this point, in which case the joint in the girder shall be offset a distance equal to twice the width of the beam. Joints in walls and columns shall be at the underside of floors, slabs, beams, or girders and at the tops of footings or floor slabs. Beams, girders, brackets, column capitals, haunches, and drop panels shall be placed at the same time as slabs. Joints shall be perpendicular to the main reinforcement.

6.1.2—All reinforcement shall be continued across joints. Longitudinal keys at least 1½ in. deep shall be provided in all joints in walls and between walls and slabs or footings. Other keys and inclined dowels shall be acceptable to the architect/engineer.

6.1.3—The surface of the concrete at all joints shall be thoroughly cleaned and all laitance removed prior to placing adjoining concrete.

6.1.4—When required or permitted, bond shall be obtained by one of the following methods:

6.1.4.1 The use of an acceptable adhesive.

6.1.4.2 The use of an acceptable chemical retarder which delays but does not prevent setting of the surface mortar. Retarded mortar shall be removed within 24 hr after placing to produce a clean exposed aggregate bonding surface.

6.1.4.3 Roughening the surface of the concrete in an acceptable manner which will expose the aggregate uniformly and will not leave laitance, loosened particles of aggregate or damaged concrete at the surface.

6.2—Expansion joints

6.2.1—Reinforcement or other embedded metal items bonded to the concrete (except dowels in floors bonded on only one side of joints) shall not be permitted to extend continuously through any expansion joint.

6.2.2—Premolded expansion joint filler shall be of the type required by the contract documents and shall conform to ASTM D 994, ASTM D 1751 or ASTM D 1752.

6.3—Waterstops

6.3.1—The material, design, and location of waterstops in joints shall be as indicated in the contract documents.

6.3.2—Each piece of premolded waterstop shall be of maximum practicable length in order that the number of end joints will be held to a minimum.

6.3.3—Joints at intersections and at ends of pieces shall be made in the manner most appropriate to the material being used. Joints shall develop effective watertightness fully equal to that of the continuous waterstop material, shall permanently develop not less than 50 percent of the mechanical strength of the parent section, and shall permanently retain their flexibility.

6.4—Other embedded items

6.4.1—All sleeves, inserts, anchors, and embedded items required for adjoining work or for its support shall be placed prior to concreting.

6.4.2—All contractors whose work is related to the concrete or must be supported by it shall be given ample notice and opportunity to introduce and/or furnish embedded items before the concrete is placed.

6.5—Placing embedded items

Expansion joint material, waterstops, and other embedded items shall be positioned accurately and supported against displacement. Voids in sleeves, inserts, and anchor slots shall be filled temporarily with readily removable material to prevent the entry of concrete into the voids.

Notes

In the sections of Chapter 6 listed below, specific acceptance is required:

6.1.1 For construction joints not indicated on the project drawings.

6.1.4 Of methods and/or procedures for developing bond at joints.

CHAPTER 7—PRODUCTION OF CONCRETE

7.1—Ready-mixed concrete and concrete produced by on-site volumetric batching and continuous mixing

7.1.1—Ready-mixed concrete shall be batched, mixed and transported in accordance with ASTM C 94, except as otherwise provided in this chapter. Plant equipment and facilities shall conform to "Certification of Ready Mixed Concrete Production Facilities (Checklist with Instructions)" of the National Ready Mixed Concrete Association.

7.1.2—Concrete produced by on-site volumetric batching and continuous mixing shall be batched and mixed in accordance with and shall conform to all requirements of ASTM C 685.

7.2—All other concretes

7.2.1 *Batching*

7.2.1.1 Scales for weighing concrete ingredients shall be accurate when in use within ±0.4 percent of their total capacities. Standard test weights shall be available to permit checking scale accuracy.

7.2.1.2 Operation of batching equipment shall be such that the concrete ingredients are consistently measured within the following tolerances:

Cement	±1 percent
Water	±1 percent
Aggregates	±2 percent
Admixtures	±3 percent

7.2.1.3 Each batch shall be so charged into the mixer that some water will enter in advance of the cement and aggregates. Water shall continue to flow for a period which may extend to the end of the first 25 percent of the specified mixing time. Controls shall be provided to prevent batched ingredients from entering the mixer before the previous batch has been completely discharged.

7.2.2 *Mixing*

7.2.2.1 The concrete shall be mixed in a batch mixer capable of thoroughly combining the aggregates, cement, and water into a uniform mass within the specified mixing time, and of discharging the concrete without harmful segregation. The mixer shall bear a manufacturer's rating plate indicating the rated capacity and the recommended revolutions per minute and shall be operated in accordance therewith.

7.2.2.2 Mixers with a rated capacity of 1 cu yd or larger shall conform to the requirements of the Plant Mixer Manufacturers Division of the Concrete Plant Manufacturers Bureau.

7.2.2.3 Except as provided in Section 7.2.2.4 below, batches of 1 cu yd or less shall be mixed for not less than 1 min. The mixing time shall be increased 15 sec for each cubic yard or fraction thereof of additional capacity.

7.2.2.4 Shorter mixing time may be permitted provided performance tests made in accordance with Section 9.3.2 of ASTM C 94 indicate that the time is sufficient to produce uniform concrete.

7.2.2.5 Controls shall be provided to insure that the batch cannot be discharged until the required mixing time has elapsed. At least three-quarters of the required mixing time shall take place after the last of the mixing water has been added.

7.2.2.6 The interior of the mixer shall be free of accumulations that will interfere with mixing action. Mixer blades shall be replaced when they have lost 10 percent of their original height.

7.3—Control of admixtures

7.3.1—Air-entraining admixtures, calcium chloride, and other chemical admixtures shall be charged into the mixer as solutions and shall be measured by means of an acceptable mechanical dispensing device. The liquid shall be considered a part of the mixing water. Admixtures that cannot be added in solution may be weighed or may be measured by volume if so recommended by the manufacturer.

7.3.2—If two or more admixtures are used in the concrete, they shall be added separately to avoid possible interaction that might interfere with the efficiency of either admixture or adversely affect the concrete.

7.3.3—Addition of retarding admixtures shall be completed within 1 min after addition of water to the cement has been completed, or prior to the beginning of the last three-quarters of the required mixing, whichever occurs first.

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7.4—Lightweight concrete

7.4.1—Lightweight aggregate concrete shall be batched and mixed as recommended by the producer of the aggregate except that, if procedures are recommended which are at variance with these specifications, they shall be acceptable.

7.4.2—Concrete made with lightweight aggregate that has been shown to absorb less than 2 percent by weight during the first hour after inundation based on test of a sample from the field-conditioned supply shall be batched and mixed as required in Sections 7.1, 7.2, and 7.3.

The "field-conditioned supply" shall consist of aggregate with the minimum moisture content likely to occur on the job. Predampening may be used to achieve this condition.

7.4.3—Concrete made with lightweight aggregates not conforming with the absorption limit of Section 7.4.2 shall be batched and mixed as follows:

7.4.3.1 The aggregate shall be added to approximately 80 percent of the mixing water and mixed for a minimum of 1½ min (15 revolutions in a truck mixer).

7.4.3.2 Then the admixtures, if any, the entire weight of cement, and the withheld portion of mixing water shall be added in the order named, and mixing completed in accordance with Section 7.1 or 7.2.2, whichever is applicable.

7.4.4—Acceptance of lightweight concrete in the field shall be based on fresh unit weight measured in accordance with Section 5.2 of ASTM C 567. The nominal fresh unit weight shall be that corresponding to the specified maximum air-dry unit weight calculated from the formula in Section 5.6 of ASTM C 567. When the nominal fresh unit weight varies more than 2 lb per cu ft from the required weight, the mixture shall be adjusted as promptly as conditions will permit to bring the unit weight to the desired level. Fresh unit weight of any batch shall not vary more than 3 lb per cu ft from the desired level.

7.5—Tempering and control of mixing water

7.5.1—Concrete shall be mixed only in quantities for immediate use. Concrete which has set shall be discarded and shall not be retempered.

7.5.2—When concrete arrives at the project with slump below that suitable for placing, as indicated by the specifications, water may be added only if neither the maximum permissible water-cement ratio nor the maximum slump is exceeded. The water shall be incorporated by additional mixing equal to at least half of the total mixing required. An addition of water above that permitted by the limitation on water-cement ratio shall be accompanied by a quantity of cement sufficient to maintain the proper water-cement ratio. Such addition shall be acceptable to the architect/engineer or his representative.

7.6—Weather conditions

7.6.1 *Cold weather*

7.6.1.1 In cold weather, the temperature of the concrete when delivered at the site of the work shall conform to the temperature limitations in Table 7.6.1.1.

7.6.1.2 If water or aggregate is heated above 100 F, the water shall be combined with the aggregate in the mixer before cement is added. Cement shall not be mixed with water or with mixtures of water and aggregate having a temperature greater than 100 F.

Table 7.6.1.1—Temperature limitations on concrete when delivered at site of work

Air temperature, deg F	Minimum concrete temperature, deg F	
	For sections with least dimension less than 12 in.	For sections with least dimension 12 in. or greater
30 to 45	60	50
0 to 30	65	55
Below 0	70	60

7.6.2 *Hot weather*

The ingredients shall be cooled before mixing, or flake ice or well-crushed ice of a size that will melt completely during mixing may be substituted for all or part of the mixing water if, due to high temperature, low slump, flash set or cold joints are encountered.

Notes

In the sections of Chapter 7 listed below, specific acceptance is required:

7.3.1 For mechanical device for dispensing admixtures.

7.4.1 To use mixing procedures for lightweight concrete that differ from those required by these specifications.

7.5.2 To add cement to mixed concrete to offset the addition of extra mixing water.

CHAPTER 8—PLACING

8.1—Preparation before placing

8.1.1—Hardened concrete and foreign materials shall be removed from the inner surfaces of the conveying equipment.

8.1.2—Formwork shall be completed; snow, ice and water shall be removed; reinforcement shall be secured in place; expansion joint material, anchors, and other embedded items shall be positioned; and the entire preparation shall be accepted.

8.1.3—Semiporous subgrades shall be sprinkled sufficiently to eliminate suction and porous subgrades shall be sealed in an acceptable manner (Section 11.1).

8.1.4—Concrete shall not be placed on frozen ground.

8.2—Conveying

8.2.1—Concrete shall be handled from the mixer to the place of final deposit as rapidly as practicable by methods which will prevent segregation or loss of ingredients and in a manner which will assure that the required quality of the concrete is maintained.

8.2.2—Conveying equipment shall be acceptable, and shall be of a size and design such that detectable setting of concrete shall not occur before adjacent concrete is placed. Conveying equipment shall be cleaned at the end of each operation or work day. Conveying equipment and operations shall conform to the following additional requirements:

8.2.2.1 Truck mixers, agitators, and nonagitating units and their manner of operation shall conform to the applicable requirements of ASTM C 94.

8.2.2.2 Belt conveyors shall be horizontal or at a slope which will not cause excessive segregation or loss of ingredients. Concrete shall be protected against undue drying or rise in temperature. An acceptable arrangement shall be used at the discharge end to prevent segregation. Mortar shall not be allowed to adhere to the return length of the belt. Long runs shall be discharged into a hopper or through a baffle.

8.2.2.3 Chutes shall be metal or metal-lined and shall have a slope not exceeding 1 vertical to 2 horizontal and not less than 1 vertical to 3 horizontal. Chutes more than 20 ft long and chutes not meeting the slope requirements may be used provided they discharge into a hopper before distribution.

8.2.2.4 Pumping or pneumatic conveying equipment shall be of suitable kind with adequate pumping capacity. Pneumatic placement shall be controlled so that segregation is not apparent in the discharged concrete. The loss of slump in pumping or pneumatic conveying equipment shall not exceed 2 in. Concrete shall not be conveyed through pipe made of aluminum or aluminum alloy.

8.3—Depositing

8.3.1 *General*—Concrete shall be deposited continuously, or in layers of such thickness that no concrete will be deposited which has hardened sufficiently to cause the formation of seams or planes of weakness within the section. If a section cannot be placed continuously, construction joints shall be located as indicated on the contract documents or as permitted. Placing shall be carried on at such a rate that the concrete which is being integrated with fresh concrete is still plastic. Concrete which has partially hardened or has been contaminated by foreign materials shall not be deposited. Temporary spreaders in forms shall be removed which the concrete placing has reached an elevation rendering their service unnecessary. They may remain embedded in the concrete only if made of metal or concrete and if prior acceptance has been obtained.

8.3.2 *Placing*—Placing of concrete in supported elements shall not be started until the concrete previously placed in columns and walls is no longer plastic and has been in place at least two hours.

8.3.3 *Segregation*—Concrete shall be deposited as nearly as practicable in its final position to avoid segregation due to rehandling or flowing. Concrete shall not be subjected to any procedure which will cause segregation.

8.3.4 *Consolidation*—All concrete shall be consolidated by vibration, spading, rodding or forking so that the concrete is thoroughly worked around the reinforcement, around embedded items, and into corners of forms, eliminating all air or stone pockets which may cause honeycombing, pitting, or planes of weakness. Internal vibrators used shall be the largest size and the most powerful that can be used properly in the work, as described in Table 5.1.5 of ACI 309R. They shall be operated by competent workmen. Use of vibrators to transport concrete within forms shall not be allowed. Vibrators shall be inserted and withdrawn at points approximately 18 in. apart. At each insertion, the duration shall be sufficient to consolidate the concrete but not sufficient to cause segregation, generally from 5 to 15 sec. A spare vibrator shall be kept on the job site during all concrete placing operations. Where the concrete is to have an as-cast finish, a full surface of mortar shall be brought against the form by the vibration process, supplemented if necessary by spading to work the coarse aggregate back from the formed surface.

8.4—Protection

8.4.1—Unless adequate protection is provided and acceptance is obtained, concrete shall not be placed during rain, sleet, or snow.

8.4.2—Rainwater shall not be allowed to increase the mixing water nor to damage the surface finish.

8.4.3 *Placing temperature*—When the temperature of the surrounding air is expected to be below 40 F during placing or within 24 hr thereafter, the temperature of the plastic concrete, as placed, shall be no lower than 55 F for sections less than 12 in. in any dimension nor 50 F for any other sections. The temperature of the concrete as placed shall not be so high as to cause difficulty from loss of slump, flash set, or cold joints and should not exceed 90 F. When the temperature of the concrete exceeds 90 F, precautionary measures acceptable to the architect/engineer shall be put into effect. When the temperature of the steel is greater than 120 F, steel forms and reinforcement shall be sprayed with water just prior to placing the concrete.

8.5—Bonding

8.5.1—When specified, the surface of joints shall be prepared in accordance with one of the methods specified in Section 6.1.4.

8.5.2—The hardened concrete of construction joints and of joints between footings and walls or columns, between walls or columns and beams or floors they support, joints in unexposed walls and all others not mentioned below shall be dampened (but not saturated) immediately prior to placing of fresh concrete.

8.5.3—The hardened concrete of horizontal construction joints in exposed work; horizontal construction joints in the middle of beams, girders, joists, and slabs; and horizontal construction joints in work designed to contain liquids shall be dampened (but not saturated) and then thoroughly covered with a coat of cement grout of similar proportions to the mortar in the concrete. The fresh concrete shall be placed before the grout has attained its initial set.

8.5.4—Joints receiving an adhesive shall have been prepared and adhesive applied in accordance with the manufacturer's recommendations prior to placing of fresh concrete.

8.5.5—Surfaces of joints which have been treated with a chemical retarder shall have been prepared in accordance with the manufacturer's recommendations prior to placing of fresh concrete.

8.6—Concreting under water

When required or permitted, concrete shall be deposited under water by an acceptable method in a way that the fresh concrete enters the mass of previously placed concrete from within, causing water to be displaced with minimum disturbance at the surface of the concrete.

Notes

In the sections of Chapter 8 listed below, specific acceptance is required:

8.1.2 Of preparation of formwork for concrete placing.

8.1.3 Of preparation of subgrade for concrete placing.

8.2.2 Of conveying equipment.

8.3.1 Of construction joint locations not indicated on the contract documents.

8.3.1 To leave temporary form spreaders, if made of metal or concrete, embedded in the concrete.

8.4.1 To place concrete during rain, sleet or snow.

8.4.3 Of precautionary measures at high temperatures.

8.6 Of the method for placing concrete under water.

CHAPTER 9—REPAIR OF SURFACE DEFECTS

9.1—General

Surface defects, including tie holes, unless otherwise specified by the contract documents, shall be repaired immediately after form removal.

9.2—Repair of defective areas

9.1.2 *Repair with portland cement mortar*

9.2.1.1 All honeycombed and other defective concrete shall be removed down to sound concrete. If chipping is necessary the edges shall be perpendicular to the surface or slightly undercut. No featheredges will be permitted. The area to be patched and an area at least 6 in. wide surrounding it shall be dampened to prevent absorption of water from the patching mortar. A bonding grout shall be prepared using a mix of approximately 1 part cement to 1 part fine sand passing a No. 30 mesh sieve, mixed to the consistency of thick cream, and then well brushed into the surface.

9.2.1.2 The patching mixture shall be made of the same materials and of approximately the same proportions as used for the concrete, except that the coarse aggregate shall be omitted and the mortar shall consist of not more than 1 part cement to 2½ parts sand by damp loose volume. White portland cement shall be substituted for a part of the gray portland cement on exposed concrete to produce a color matching the color of the surrounding concrete, as determined by a trial patch. The quantity of mixing water shall be no more than necessary for handling and placing. The patching mortar shall be mixed in advance and allowed to stand with frequent manipulation with a trowel, without addition of water, until it has reached the stiffest consistency that will permit placing.

9.2.1.3 After surface water has evaporated from the area to be patched, the bond coat shall be well brushed into the surface. When the bond coat begins to lose the water sheen, the premixed patching mortar shall be applied. The mortar shall be thoroughly consolidated into place and struck off so as to leave the patch slightly higher than the surrounding surface. To permit initial shrinkage, it shall be left undisturbed for at least 1 hr before being finally finished. The patched area shall be kept damp for 7 days. Metal tools shall not be used in finishing a patch in a formed wall which will be exposed.

9.2.2—Repair materials and procedures, other than those specified in Section 9.2.1, may be used for repair when acceptable. Materials include but are not limited to:

9.2.2.1 Shotcrete.

9.2.2.2 Commercial patching products, including:

(a) Latex-modified portland cement mortar.

(b) Latex bonding agents if not re-emulsifiable when subsequently exposed to moisture.

(c) Epoxy mortars and compounds that are moisture insensitive with an epoxy binder that conforms to ASTM C 881, Type III.

Caution shall be exercised when using these materials with regard to possible color changes from weathering and delamination due to different coefficients of thermal expansion.

9.2.3—When required for exposed concrete that will be left unpainted, color tests shall be made with patching methods and materials to determine color compatibility.

9.3—Tie holes

Tie holes shall be plugged unless stainless steel, noncorrosive, or acceptably coated ties are used. When portland cement mortar is used for plugging, tie holes shall be cleaned and dampened prior to patching. When surface is to be textured for architectural appearance by sandblasting or bushhammering, minor defects, and tie holes shall be repaired to match the adjoining concrete in color and texture when viewed from a distance of 15 ft. Other materials used for plugging the holes shall be subject to acceptance by the architect/engineer and shall be applied in accordance with the manufacturer's written recommendations, where applicable.

9.4—Removal of stains, rust, efflorescence, and surface deposits

Stains, rust, efflorescence, and surface deposits considered objectionable by the architect/engineer shall be removed by methods acceptable to the architect/engineer.

Notes

In the Sections of Chapter 9 listed below, acceptance is required:

9.2 To use specific patching products, materials, and proportions.

9.2.2.2(c) To use epoxy materials and compounds, with regard to possible color changes and delamination of thin repairs.

9.2.3 To use specific materials, adhesives, and prefabricated plugging devices for plugging tie holes.

9.2.4 For methods to be used for removal of stains, rust, efflorescence, and surface deposits.

CHAPTER 10—FINISHING OF FORMED SURFACES

10.1—General

10.1.1—After removal of forms the surfaces of concrete shall be given or more of the finishes specified below in locations designated by the contract documents or as specified in Section 10.4.

10.1.2—When finishing is required to match a small sample furnished to the contractor, the sample finish shall be reproduced on an area at least 100 sq ft in an inconspicuous location designated by the architect/engineer before proceeding with the finish in the specified location.

10.2—As-cast finishes

10.2.1 *Rough form finish*—No selected form facing materials shall be specified for rough form finish surfaces. Tie holes and defects shall be patched. Fins exceeding ¼ in. in height shall be chipped off or rubbed off. Otherwise, surfaces shall be left with the texture imparted by the forms.

10.2.2 *Smooth form finish*—The form facing material shall produce a smooth, hard, uniform texture on the concrete. It may be plywood, tempered concrete-form-grade hardboard, metal, plastic, paper, or other acceptable material capable of producing the desired finish. The arrangement of the facing material shall be orderly and symmetrical, with the number of seams kept to the practical minimum. It shall be supported by studs or other backing capable of preventing excessive deflection (see Table 4.3.1 for tolerances). Material with raised grain, torn surfaces, worn edges, patches, dents, or other defects which will impair the texture of the concrete surface shall not be used. Tie holes and defects shall be patched. All fins shall be completely removed.

10.2.3 *Special architectural finishes*—Textured finish, exposed aggregate finish, and aggregate transfer finish are architectural concrete finishes and shall be produced in accordance with the requirements of Chapter 13, Architectural Concrete.

10.3—Rubbed finishes

The following finishes shall be produced on concrete with a smooth form finish (Section 10.2.2). Where smooth rubbed finish is to be applied, the forms shall have been removed and necessary patching completed as soon after placement as possible without jeopardizing the structure.

10.3.1 *Smooth rubbed finish*—Smooth rubbed finish shall be produced on newly hardened concrete no later than the day following form removal. Surfaces shall be wetted and rubbed with carborundum brick or other abrasive until uniform color and texture are produced. No cement grout shall be used other than the cement paste drawn from the concrete itself by the rubbing process.

10.3.2 *Grout cleaned finish*—No cleaning operations shall be undertaken until all contiguous surfaces to be cleaned are completed and accessible. Cleaning as the work progresses shall not be permitted. Mix 1 part portland cement and 1½ parts fine sand with sufficient water to produce a grout having the consistency of thick paint. White portland cement shall be substituted for a part of the gray portland cement in order to produce a color matching the color of the surrounding concrete, as determined by a trial patch. Wet the surface of the concrete sufficiently to prevent absorption of water from the grout and apply the grout uniformly with brushes or a spray gun. Immediately after applying the grout, scrub the surface vigorously with a cork float or stone to coat the surface and fill all air bubbles and holes. While the grout is still plastic, remove all excess grout by working the surface with a rubber float, burlap, or other means. After the surface whitens from drying (about thirty minutes at normal temperatures), rub vigorously with clean burlap. The finish shall be kept damp for at least 36 hours after final rubbing.

10.3.3 *Cork floated finish*—Remove forms at an early stage, within 2 to 3 days of placement where possible. Remove ties. Remove all burrs and fins. Mix one part portland cement and one part fine sand with sufficient water to produce a stiff mortar. Dampen wall surface. Apply mortar with firm rubber float or with trowel, filling all surface voids. Compress mortar into voids using a slow-speed grinder or stone. If the mortar surface dries too rapidly to permit proper compaction and finishing, apply a small amount of water with a fog sprayer. Produce the final texture with a cork float using a swirling motion.

10.4—Unspecified finish

If the finish is not designated in the contract documents, the following finishes shall be used as applicable:

10.4.1 *Rough form finish*—For all concrete surfaces not exposed to public view.

10.4.2 *Smooth form finish*—For all concrete surfaces exposed to public view.

10.5—Rela[illegible] unformed surfaces

Tops of walls or buttresses, horizontal offsets, and similar unformed surfaces occurring adjacent to formed surfaces shall be struck smooth after concrete is placed and shall be floated to a texture reasonably consistent with that of the formed surfaces. Final treatment on formed surfaces shall continue uniformly across the unformed surfaces.

Notes

In the sections of Chapter 10 listed below, acceptance must be obtained:

10.1.2 Of experimental surface finish required to match a sample.

10.2.2 For facing materials other than those enumerated.

CHAPTER 11—SLABS

11.1—General

Concrete for slabs shall be proportioned in accordance with Section 3.14 to meet the requirements for the class of floor designated in the contract documents.

11.2—Preparation of subgrade for slabs on ground

11.2.1—The subgrade shall be well drained and of adequate and uniform loadbearing capacity. The minimum in-place density of the subgrade soils shall be as required in the specifications. The bottom of an *undrained* granular base course shall not be lower than the adjacent finished grade.

11.2.2—The subgrade shall be free of frost before concrete placing begins. If the temperature inside a building where concrete is to be placed is below freezing it shall be raised and maintained above 50 F long enough to remove all frost from the subgrade.

11.2.3—The subgrade shall be moist at the time of concreting. If necessary, it shall be dampened with water in advance of concreting, but there shall be no standing water on the subgrade nor any muddy or soft spots when the concrete is placed.

11.3—Edge forms and screeds

11.3.1—Edge forms and intermediate screed strips shall be set accurately to produce the designated elevations and contours of the finished surface, and shall be sufficiently strong to support vibrating screeds or roller pipe screeds if the nature of the finish specified requires the use of such equipment. The concrete surface shall be aligned to the contours of screed strips by the use of strike-off templates or acceptable compacting type screeds.

11.3.2—When formwork is cambered, screeds shall be set to a like camber to maintain the proper concrete thicknesses.

11.4—Placement

11.4.1—Mixing and placing shall be carefully coordinated with finishing. Concrete shall not be placed on the subgrade or forms more rapidly than it can be spread, straightedged, and darbied or bull floated. These operations must be performed before bleeding water has an opportunity to collect on the surface.

11.4.2—To obtain good surfaces and avoid cold joints, the size of finishing crews shall be planned with due regard for the effects of concrete temperature and atmospheric conditions on the rate of hardening of the concrete. If construction joints become necessary, they shall be constructed as required in Chapter 6.

11.5—Jointing

Joints in slabs on grade shall be located and detailed as indicated in the contract documents. If saw-cut joints are required or permitted, cutting shall be timed properly with the set of concrete. Cutting shall be started as soon as the concrete has hardened sufficiently to prevent aggregates being dislodged by the saw. Cutting shall be completed before shrinkage stresses become sufficient to produce cracking.

11.6—Consolidation

Concrete in slabs shall be thoroughly consolidated. Internal vibration shall be used in beams and girders of framed slabs, and along the bulkheads of slabs on grade. Consolidation of slabs shall be obtained with vibrating screeds, roller pipe screeds, internal vibrators, or other acceptable means.

11.7—Finishes (see Section 11.9 for definitions of tolerance classes)

11.7.1 *Scratched finish*—After the concrete has been placed, consolidated, struck off, and leveled to a Class C tolerance, the surface shall be roughened with stiff brushes or rakes before the final set.

11.7.2 *Floated finish*—After the concrete has been placed, consolidated, struck off, and leveled, the concrete shall not be worked further until ready for floating. Floating with a hand float or with a bladed power trowel equipped with float shoes, or with a powered disc float shall begin when the water sheen has disappeared and when the surface has stiffened sufficiently to permit the operation. During or after the first floating, planeness of surface shall be checked with a 10-ft straightedge applied at not less than two different angles. All high spots shall be cut down and all low spots filled during this procedure to produce a surface within Class B tolerance throughout. The slab shall then be refloated immediately to a uniform sandy texture.

11.7.3 *Troweled finish*—The surface shall first be float-finished as specified in Section 11.7.2. It shall next be power troweled, and finally hand troweled. The first troweling after power floating shall produce a smooth surface which is relatively free of defects but which may still show some trowel marks. Additional trowelings shall be done by hand after the surface has hardened sufficiently. The final troweling shall be done when a ringing sound is produced as the trowel is moved over the surface. The surface shall be thoroughly consolidated by the hand troweling operations. The finished surface shall be essentially free of trowel marks, uniform in texture and appearance and shall be plane to a Class A tolerance, except tolerance for concrete on metal deck shall be Class B. On surfaces intended to support floor coverings, any defects of sufficient magnitude to show through the floor covering shall be removed by grinding.

11.7.4 *Broom or belt finish*—Immediately after the concrete has received a float finish as specified in Section 11.7.2, it shall be given a coarse transverse scored texture by drawing a broom or burlap belt across the surface.

11.7.5 *Heavy-duty topping for two-course slabs*

11.7.5.1 The topping mixture shall be composed of materials selected to impart heavy-duty wearing properties to the finished slab. Materials selected shall be acceptable to the architect/engineer.

11.7.5.2 The base slab shall be placed and screeded ¾ to 1 in. below the required finish surface and consolidated. The concrete shall not be worked further until ready for the next operation.

11.7.5.3 Topping placed on the same day as base slab. When bleed water has disappeared and the surface of the base slab will support a man without appreciable indentation, the topping mixture shall be spread, compacted, floated, checked for trueness of surface, and finished in the manner specified above for floated finish (see Section 11.7.2) or troweled finish (see Section 11.7.3) except that power-driven floats shall be of the impact type.

11.7.5.4 Topping placement deferred. As soon as the base slab has partially set, the surface shall be brushed with a coarse wire broom to remove laitance and scratch the surface. The base slab shall be wet cured a minimum of three days. The scratched base slab shall be protected from contamination until time to place the topping. Prior to placement of topping, the base slab shall be thoroughly cleaned and dampened but left free of standing water. Immediately before the topping is placed, a coat of bonding grout prepared in accordance with Section 9.2.1 shall be scrubbed into the surface; it shall not be allowed to set or dry before the topping is placed. Bonding agents other than cement grout may be used if acceptable. The topping mixture shall be placed, compacted, and finished as described in Section 11.7.5.3.

11.7.6 *"Dry shake" finish*—If a "dry shake" application of a selected metallic or mineral aggregate is specified, the aggregate, selected or acceptable to the architect/engineer, shall be blended with portland cement in the proportions recommended by the manufacturer of the aggregate. The surface shall be given a float finish in accordance with Section 11.7.2. Approximately two-thirds of the blended material for required coverage shall be applied to the surface by a method that insures even coverage without segregation. Floating shall begin immediately after application of the first "dry shake." After this material has been embedded by floating, the remainder of the blended material shall be applied to the surface at right angles to the previous application. The second application shall be heavier in any areas not sufficiently covered by the first application. A second floating shall follow immediately. After the selected material has been embedded by the two floatings, the operation shall be completed with a broomed, floated, or troweled finish, as designated in the contract documents.

11.7.7 *Nonslip finish*—Where the contract documents require a nonslip finish, the surface shall be given a "dry shake" application, as specified above, of crushed ceramically bonded aluminum oxide or other specified selected abrasive particles. The rate of application of such material shall be not less than 25 lb per 100 sq ft.

11.7.8 *Topping for two-course slab not intended for heavy duty service*—Preparation of base slab, selection of topping material, mixing, placing, consolidating and finishing operations shall be as specified in Section 11.7.5 above, except that the aggregate need not be selected for special wear resistance.

11.7.9 *Exposed aggregate finish*—Immediately after the surface of the concrete has been leveled to a Class B tolerance in accordance with Section 11.7.2 and surface water has disappeared, aggregate of color and size (usually ⅜ to ⅝ in.) selected by the architect/engineer shall be spread uniformly over the surface to provide complete coverage to the depth of a single stone.

11.7.9.1 The spread of selected aggregate shall be embedded into the surface by light tamping and the surface shall be floated until the embedded stone is fully coated with mortar and the surface has been brought to a true plane within Class B tolerance. Exposure of the aggregate shall start after the matrix has hardened sufficiently to prevent dislodgement of the aggregate. Water, in abundant quantities but without force, shall be allowed to flow over the surface of the concrete while the matrix encasing the selected aggregate is removed by brushing with a fine bristle brush. This operation shall continue until the selected aggregate is uniformly exposed but not dislodged.

11.7.9.2 An acceptacle chemical retarder sprayed onto the freshly floated surface may be used to extend the working time for exposure of aggregate.

11.8—Unspecified finish

When type of finish is not specified in the contract documents, the following finishes shall be used as applicable:

11.8.1 *Scratched finish*—For surfaces intended to receive bonded applied cementitious applications.

11.8.2 *Floated finish*—For surfaces intended to receive roofing, waterproofing membranes, or sand bed terrazzo.

11.8.3 *Troweled finish*—For floor intended as walking surfaces or for reception of floor coverings.

11.8.4 *Broom or belt finish*—For sidewalks and garage floors and ramps.

11.8.5 *Nonslip finish*—For exterior platforms, steps, and landings; and for exterior and interior pedestrian ramps.

11.9—Finishing tolerances

11.9.1—Finishes with Class A tolerances shall be true planes within ⅛ in. in 10 ft, as determined by a 10-ft straightedge placed anywhere on the slab in any direction.

11.9.2—Finishes with Class B tolerances shall be true planes within ¼ in. in 10 ft, as determined by a 10-ft straightedge placed anywhere on the slab in any direction.

11.9.3—Finishes with Class C tolerances shall be true planes within ¼ in. in 2 ft as determined by a 2-ft straightedge placed anywhere on the slab in any direction.

Notes

In the sections of Chapter 11 listed below, specific acceptance is required:

11.3.1 Of compacting type screeds.

11.6 Of special methods for consolidation of concrete.

11.7.5.1 Of materials for heavy duty topping mixture.

11.7.5.4 Of bonding agents other than grout.

11.7.6 Of material for "dry shake" application.

11.7.9.2 Of chemical retarder used to extend working time in producing exposed aggregate finish.

CHAPTER 12—CURING AND PROTECTION

12.1—General

Beginning immediately after placement, concrete shall be protected from premature drying, excessively hot or cold temperatures, and mechanical injury, and shall be maintained with minimal moisture loss at a relatively constant temperature for the period necessary for hydration of the cement and hardening of the concrete. The materials and methods of curing shall be subject to acceptance.

12.2—Preservation of moisture

12.2.1—For concrete surfaces not in contact with forms, one of the following procedures shall be applied immediately after completion of placement and finishing:

12.2.1.1 Ponding or continuous sprinkling.

12.2.1.2 Application of absorptive mats of fabric kept continuously wet.

12.2.1.3 Application of sand kept continuously wet.

12.2.1.4 Continuous application of steam (not exceeding 150 F) or mist spray.

12.2.1.5 Application of waterproof sheet materials, conforming to ASTM C 171.

12.2.1.6 Application of other acceptable moisture-retaining covering.

12.2.1.7 Application of a curing compound conforming to ASTM C 309. The compound shall be applied in accordance with the recommendations of the manufacturer immediately after any water sheen which may develop after finishing has disappeared from the concrete surface. It shall not be used on any surface against which additional concrete or other material is to be bonded unless it is proven that the curing compound will not prevent bond, or unless positive measures are taken to remove it completely from areas to receive bonded applications.

12.2.2—Moisture loss from surfaces placed against wooden forms or metal forms exposed to heating by the sun shall be minimized by keeping the forms wet until they can be safely removed. After form removal the concrete shall be cured until the end of the time prescribed in Section 12.2.3 by one of the methods of Section 12.2.1.

12.2.3—Curing in accordance with Section 12.2.1 or 12.2.2 shall be continued for at least 7 days in the case of all concrete except high-early-strength concrete for which the period shall be at least 3 days. Alternatively, if tests are made of cylinders kept adjacent to the structure and cured by the same methods, moisture retention measures may be terminated when the average compressive strength has reached 70 percent of the specified strength, f'_c. Moisture retention measures may also be terminated when the temperature of the concrete is maintained at least at 50 F for the same length of time that laboratory-cured cylinders, representative of the concrete-in-place, require to achieve 85 percent of f'_c. If one of the curing procedures of Sections 12.2.1.1 through 12.2.1.4 is used initially, it may be replaced by one of the other procedures of Section 12.2.1 any time after the concrete is 1 day old provided the concrete is not permitted to become surface dry during the transition.

12.3—Temperature, wind, and humidity

12.3.1 *Cold weather*—When the mean daily outdoor temperature is less than 40 F, the temperature of the concrete shall be maintained between 50 and 70 F for the required curing period of Section 12.2.3. When necessary, arrangements for heating, covering, insulating, or housing the concrete work shall be made in advance of placement and shall be adequate to maintain the required temperature without injury due to concentration of heat. Combustion heaters shall not be used during the first 24 hr unless precautions are taken to prevent exposure of the concrete to exhaust gases which contain carbon dioxide.

12.3.2 *Hot weather*—When necessary, provision for windbreaks, shading, fog spraying, sprinkling, ponding, or wet covering with a light colored material shall be made in advance of placement, and such protective measures shall be taken as quickly as concrete hardening and finishing operations will allow.

12.3.3 *Rate of temperature change*—Changes in temperature of the air immediately adjacent to the concrete during and immediately following the curing period shall be kept as uniform as possible and shall not exceed 5 F in any 1 hr or 50 F in any 24-hr period.

12.4—Protection from mechanical injury

During the curing period, the concrete shall be protected from damaging mechanical disturbances, such as load stresses, heavy stock, and excessive vibration. All finished concrete surfaces shall be protected from damage by construction equipment, materials, or methods, by application of curing procedures, and by rain or running water. Self-supporting structures shall not be loaded in such a way as to overstress the concrete.

Notes

In the sections of Chapter 12 listed below, specific acceptance is required:

12.1 Of materials and methods of curing.

12.2.1.6 To use moisture-retaining covering of a type not listed.

CHAPTER 13—ARCHITECTURAL CONCRETE

13.1—General

Architectural concrete is concrete which is exposed to view as an interior or exterior surface in the completed structure, and specifically designated as such in the contract documents.

13.2—Proportioning

13.2.1—Unless the contract documents require a plaster coat finish or final painting of surfaces, designated colors and uniformity of color shall be maintained. For concrete of a desired color, the same mixture proportions shall be used throughout. Changes in the quantity of portland cement per cubic yard shall be particularly avoided. Only one type and one brand of cement from the same mill, only one source and one maximum size of coarse aggregate, only one source of fine aggregate, and only one placing consistency shall be used.

13.2.2—Architectural concrete for exterior exposure shall be air-entrained with a water-cement ratio not exceeding 0.46 by weight.

13.3—Forms

13.3.1—Forms for architectural concrete shall be designed to produce the required finish or finishes. Deflection of facing materials between studs as well as deflection of studs and walers shall be limited to 0.0025 times the span or as otherwise specified. Forms shall be designed to permit easy removal. Prying against the face of the concrete shall not be allowed. Only wooden wedges shall be used.

13.3.2—Where natural plywood form finish, grout cleaned finish, smooth rubbed finish, scrubbed finish, or sand floated finish is required, forms shall be smooth (faced with plywood, liner sheets, or prefabricated panels) and true to line, in order that the surfaces produced will require little dressing to arrive at true surfaces. Where any as-cast finish is required, no dressing shall be permitted in the finishing operation.

13.3.3—Where as-cast surfaces, including natural plywood form finish, are specified, the panels of material against which concrete is cast shall be orderly in arrangement, with joints between panels planned in acceptable relation to openings, building corners, and other architectural features.

13.3.4—Where panels for as-cast surfaces are separated by recessed or otherwise emphasized joints, the structural design of the forms shall provide for locating form ties, where possible, within the joints so that patches of tie holes will not fall within the panel areas.

13.3.5—In addition to shop drawings normally required, fabricating drawings of forms for architectural concrete shall be submitted for acceptance showing the jointing of facing panels, the locations of form ties, and any necessary alignment bracing.

13.3.6—Forms shall not be reused if there is any evidence of surface wear and tear or defect which would impair the quality of the surface. Forms shall be thoroughly cleaned and properly coated before reuse.

13.3.7—Formwork for architectural concrete shall be observed continuously while concrete is being placed to see that there are no deviations from desired elevation, alignment, plumbness, or camber. If, during construction, any weakness develops and the falsework shows any undue settlement or distortion, the work shall be stopped, the affected construction removed if permanently damaged, and the falsework strengthened.

13.4—Placing of concrete

13.4.1—Where a smooth rubbed or similar finish is required, the coarse aggregate shall be worked back from the forms, leaving a full surface of mortar but avoiding the production of surface voids.

13.4.2—Vibrators shall not be allowed to contact the formwork for exposed concrete surfaces.

13.5—Special architectural finishes

13.5.1 *Textured finishes*—Textured form liners may be of formed plastic sheet, wood, sheet metal, or other material designated in the contact documents. Liner panels shall be secured in forms by cementing or stapling, but not by methods which will permit impressions of nail heads, screw heads, washers, or the like to be imparted to the surface of the concrete. Edges of textured panels shall be sealed to each other or to divider strips (if specified or shown) to prevent bleeding of grout. The sealant used shall be nonstaining to the surface.

13.5.2 *Aggregate transfer finishes*—Aggregate transfer and other special finishes shall be produced using methods and materials designated in the contract documents in such a way as to duplicate sample panels prepared in advance.

13.5.3 *Exposed aggregate finishes*—Aggregate shall be exposed by a method, acceptable to the architect/engineer, such as sandblasting, bushhammering, or the use of a surface retarder. The surface shall be produced in such a way as to duplicate a sample panel prepared in advance. The contractor shall submit to the architect/engineer for acceptance, prior to placement, the intended procedure, such as use of gap-graded mixtures or preplaced aggregates or other, by means of which uniform distribution of the exposed aggregate will be achieved.

13.5.3.1 Scrubbed finish. Scrubbed finish shall be produced on partially-hardened concrete. The surface shall be thoroughly wetted and scrubbed with stiff fiber or wire brushes, using water freely, until the surface mortar is removed and the aggregate is uniformly exposed. The surface shall then be rinsed with clean water. If portions of the surface have become too hard to permit uniform aggregate exposure, dilute hydrochloric acid (commercial muriatic acid diluted with 4 to 10 parts water) may be used to remove the excess after the concrete is at least 2 weeks old. The acid shall be removed from the finished surface with clean water within 15 min after application.

Aggregate exposure may be facilitated by casting the concrete against form faces which have been coated with a chemical retarder used in accordance with the manufacturer's recommendations to keep the mortar adjacent to the form from setting.

13.5.3.2 Sand blast finish. The concrete surface shall be sandblasted. Unless otherwise specified, degree of sandblasting shall be light. [Light sandblasting is sandblasting sufficient to expose fine aggregate with occasional exposure of coarse aggregate; exposed coarse aggregate should not project more than 1/16 in. (1.5 mm) from the matrix.]

13.5.3.3 Tooled finish. The thoroughly cured concrete surface shall be dressed with electric, air, or hand tools to a uniform texture, and shall be given a hand tooled, rough or fine pointed, crandalled, or bushhammered surface texture, as designated by the contract documents.

13.5.3.4 If finishes are specified in accordance with either Section 13.5.3.2 or 13.5.3.3, the depth of penetration of the finish shall be specified by one of the following criteria:

13.5.3.4.1 Remove only the surface mortar.

13.5.3.4.2 Remove sufficient mortar to expose the surface of some coarse aggregate.

13.5.3.4.3 Remove sufficient mortar to expose the coarse aggregate in relief to the specified depth (for sand blast) or to fracture the coarse aggregate (for tooled finish).

13.5.4 *Applied finishes*—When finishes of stucco or similar troweled materials are to be applied, the surface of the concrete shall be prepared to insure permanent adhesion of the finish. If the concrete is less than about 24 hr old, it can be roughened with a heavy wire brush or scoring tool. If the concrete is older the surface may be roughened mechanically as specified in Section 13.5.3.2 or 13.5.3.3 or by etching with acid as specified in Section 13.5.3.1. After roughening, the surface shall be washed free of all dust, acid, chemical retarder, and other foreign material before the final finish is applied.

13.6—Patching

13.6.1—Where as-cast finishes are specified, the total patched area shall not exceed 2 sq ft in each 1000 sq ft of as-cast surface. This is in addition to form tie patches, if the contract documents permit ties to fall within as-cast areas.

13.6.2—Any patches in as-cast architectural concrete shall closely match the color and texture of surrounding surfaces. The mix formula for patching mortar shall be determined by trial to obtain a good color match with the concrete when both patch and concrete are cured and dry. After initial set, surfaces of patches shall be dressed manually to obtain the same texture as surrounding surfaces.

13.6.3—In any finishing process which is intended to expose aggregate on the surface, patched areas shall show aggregate faces. The outer 1 in. of patch shall contain the same aggregates as the surrounding concrete. In the case of aggregate transfer finish, the patching mixture shall contain the same selected colored aggregates. After patches have been allowed to cure thoroughly, the aggregates shall be exposed together with the aggregates of adjoining surfaces by the same process of mortar removal.

13.6.4—Patches in architectural concrete surfaces shall be cured for 7 days. Patches shall be protected from premature drying to the same extent as the body of the concrete.

Notes

In the sections of Chapter 13 listed below, specific acceptance is required:

13.3.3 Of form joint locations between panels.

13.3.5 Of drawings for formwork.

13.5.3 Of the method for developing exposed aggregate finish, and the means by which uniform distribution of the aggregate will be achieved.

CHAPTER 14—MASSIVE CONCRETE

14.1—General

14.1.1—Portions of the structure to be treated as massive concrete under the provisions of this chapter shall be designated in the contract documents.

14.1.2—Such massive concrete shall be subject to the requirements of this chapter in addition to all other applicable provisions of these Specifications.

14.2—Materials

14.2.1—High-early-strength (Type III) cement, calcium chloride, and accelerating type admixtures shall not be used.

14.2.2—A retarding admixture, pretested with job materials under job conditions, shall be used, if acceptable, whenever necessary to prevent cold joints due to the quantity of concrete placed, to permit revibration of the concrete, to offset the effects of high concrete temperature, or to reduce the maximum temperature and rate of temperature rise.

14.3—Proportioning

Cement content shall be the minimum required to attain the specified compressive strength f'_c, durability and other specified properties.

14.4—Placing

14.4.1—Unless otherwise permitted or specified, the slump of unreinforced concrete or of massive concrete that contains no greater reinforcement than required for temperature and shrinkage shall be 3 in. or less. A tolerance of up to 1 in. above the maximum indicated shall be allowed for one batch in any five consecutive batches tested. The slump of other reinforced massive concrete shall comply with Section 3.5.

14.4.2—The maximum temperature of the concrete when deposited shall be 70 F.

14.4.3—Concrete shall be placed in layers approximately 18 in. thick. Vibrator heads shall extend into the previously placed layer.

14.5—Curing and protection

14.5.1—The minimum curing period shall be 2 weeks.

14.5.2—When the surrounding air temperature falls below 32 F, the surface of the concrete shall be protected against freezing but steam or other curing methods that will add heat to the concrete shall not be used.

14.5.3—When the surrounding air temperature is greater than 40 F, the forms and exposed concrete shall be kept continuously wet for the first 48 hours after placing. Such wetting is also required whenever the surrounding air temperature exceeds 90 F during the remainder of the curing period. When the surrounding air temperature is less than 40 F, the concrete shall be protected from freezing and moisture loss, but continuous wetting during the first 48 hours is not required.

14.5.4—During and at the conclusion of the specified curing period, means shall be provided to insure that the temperature of the air immediately adjacent to the concrete does not fall more than 3 F in any 1 hr nor more than 30 F in any 24 hr.

Note

In Section 14.2.2, specific acceptance is required to use a retarded admixture to facilitate placement of massive concrete.

CHAPTER 15—PRESTRESSED CONCRETE

15.1—General

15.1.1—Job-cast, post-tensioned, prestressed structural members, except tension members, shall conform to the special provisions of this chapter in addition to all applicable provisions of other chapters in these specifications.

15.1.2 *Definitions*

Anchorage—A device used to anchor the tendon to the concrete member.

Bonded tendon—A prestressing tendon which is bonded to the concrete either directly or through grouting.

Coating—Material applied to unbonded tendons to protect them from corrosion; or material applied to either bonded or unbonded tendons to lubricate them during stressing.

Coupling—Any device designed to transfer the prestressing force from one tendon to another.

Element diameter—The diameter of the individual prestressing steel wires, bars or strands that comprise a tendon.

Prestressing steel—That element of a post-tensioning tendon which is elongated and anchored to provide the necessary permanent prestressing force.

Sheathing—An enclosure in which post-tensioned tendons are encased to prevent bonding during concrete placement, such as a paper or plastic jacket for unbonded tendons, or metal conduit for bonded tendons.

Tendon—An assemblage of steel elements such as wire, bar, or strand, complete with anchorages or anchorage devices used to impart prestress to concrete when the assembly is tensioned.

Unbonded tendon—A tendon which is not bonded to the concrete.

15.2—Materials

15.2.1 *Prestressing steel*

15.2.1.1 Prestressing steel shall be of the type and strength required by the contract documents and shall conform to the appropriate one of the following specifications:

(a) ASTM A 416
(b) ASTM A 421
(c) ASTM A 722

15.2.1.2 Strands, wire and bars not specifically listed in ASTM A 416, A 421, or A 722 may be used provided they conform to the minimum requirements of these specifications and do not have properties that make them less satisfactory than those listed in ASTM A 416, A 421 or A 722.

15.2.1.3 The typical stress-strain curve of the prestressing steel shall be submitted for review. An actual curve representing the production lot from which the project material was taken may be required to verify its compliance with the typical curve. For materials not produced under an ASTM specification, the guaranteed ultimate strength, yield strength, elongation, composition, and other pertinent data shall be submitted. Certified mill test reports shall be submitted for review when requested.

15.2.1.4 The amount of stress loss normally expected in seating anchorage devices, the friction wobble coefficient, and friction curvature coefficient expected for the tendons and duct-forming material, shall be submitted for review. If requested, acceptable test data substantiating the expected coefficients and anchorage slip shall be submitted for review.

15.2.1.5 Tendons shall be clean and free of excessive rust, scale, and pitting. A light oxide coating is permissible. Unbonded tendons shall be protected against corrosion by an acceptable coating such as epoxy, grease, wax, plastic, or bituminous material. Throughout the anticipated range of temperatures for the structure, the coating material shall remain ductile and free from cracks and shall not become fluid. The coating shall be chemically non-reactive with the tendon, concrete, and the material used for sheathing. The coating shall adhere to and be continuous over the entire tendon length to be unbonded. Where tendons are outside of the concrete of the post-tensioned element or where the structure is in or exposed to an atmosphere of salt air or high humidity, an additional field-applied coating of acceptable material shall be applied.

15.2.1.6 Tendons shall not be subjected to excessive temperatures, welding sparks, or electric ground currents. To insure that this requirement is met, burning and welding operation shall not be conducted in the vicinity of tendons without prior acceptance. Superfluous extension of tendons beyond anchorages may be removed by rapid oxyacetylene burning, unless such procedures are contrary to the recommendations of the manufacturer of the prestressing steel.

15.2.2 *Anchorages and couplings*

15.2.2.1 Bonded tendon anchorages tested in an unbonded state, shall develop 90 percent of the minimum specified ultimate strength of the prestressing steel, without exceeding anticipated set at time of anchorage, and without slip. Anchors which develop less than 100 percent of the minimum specified ultimate strength shall be applied only where the bond length is equal to, or greater than the bond length required to develop 100 percent of the minimum specified ultimate strength of the tendon. The required bond length shall be provided between the anchorage and the zone where the full prestressing force will be developed under service and ultimate loads. The bond length shall be determined by testing a full-sized tendon. If, in the unbonded state, the anchorage develops 100 percent of the minimum specified ultimate strength, it need not be tested in the bonded state.

15.2.2.2 Unbonded tendon anchorages shall develop the minimum specified ultimate strength of the prestressing steel with a minimum amount of permanent deformation which will not decrease the expected ultimate strength. The total elongation under ultimate load of the tendon shall not be less than 2 percent when measured over a minimum gage length of 10 ft. Evidence shall be submitted for review, demonstrating compliance with the dynamic test of Section 15.2.3.

15.2.2.3 Couplings shall be used only at locations specifically indicated on the contract documents or as acceptable. All couplings shall develop the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set of either the coupling or the prestressing steel, and shall not reduce the ductility of the tendon below the minimum 2 percent elongation as specified in Section 15.2.2.2. Couplings shall be enclosed in sheaths which permit necessary movements during stressing. For bonded tendons, fittings shall be provided to allow complete grouting of all the coupling components.

15.2.2.4 When requested, satisfactory test data in accordance with Section 15.2.3 confirming the adequacy of the proposed anchorages and couplings shall be submitted for review. Tendons composed of multiple strands, wires or bars in a common sheath should be tensioned, simultaneously unless the effects of interferences between the elements are considered.

15.2.3 *Qualification for post-tensioning systems*—The tendon shall be subjected to testing to verify its adequacy prior to use. These tests shall be performed according to the following requirements:

15.2.3.1 Test assembly. Two samples of each tendon size at least 10 ft in length and complete with standard production quality anchorages shall be tested in accordance with Section 15.2.3.2 (and when required a third such sample shall be tested in accordance with Section 15.2.3.3). Testing procedures and apparatus shall be arranged to simulate field conditions as nearly as possible and anchorages shall be seated or positioned using field procedures and equipment.

15.2.3.2 Static test. The prestressing steel samples shall be tested in accordance with the appropriate specification of Section 15.2.1. The tendon assembly shall be tested in such a manner as to allow accurate determination of the yield strength, ultimate strength and elongation of the specimen to insure compliance with Section 15.2.2.

15.2.3.3 Dynamic testing. For unbonded tendons, a dynamic test shall be performed on a representative tendon assembly and shall withstand without failure 500,000 cycles from 60 to 66 percent of its guaranteed minimum ultimate strength. One cycle involves the change from 60 to 66 percent and return to 60 percent. A prototype tendon assembly may be used provided that it has not less than 10 percent of the full size tendon strength. Single element tendons utilizing one strand, bar or wire steel shall be tested as a complete tendon assembly. Systems utilizing multiple strands, wires or bars may be tested using a prototype tendon of sufficient number of elements to duplicate the behavior of a full-sized tendon.

15.2.4 *Sheathing for bonded tendons*

15.2.4.1 Sheathing or duct-forming material shall be of material that will not react with alkalies in the cement, strong enough to retain its shape and resist damage during construction. It shall prevent the entrance of cement paste from the concrete. Sheathing material left in place shall not cause electrolytic action or deterioration.

15.2.4.2 The inside diameter of the sheathing shall be at least ¼ in. larger than the wire, bar or strand tendon and shall have an inside cross-sectional area at least twice that of the net area of the tendon.

15.2.4.3 Sheathing shall have grout holes or vents at each end and at all high points except where curvature is small and the sheathing is relatively level such as in continuous slabs. Drain holes shall be provided at all low points if the tendon may be subjected to freezing after placing and before grouting.

15.2.5 *Sheathing for unbonded tendons*

15.2.5.1 The sheathing shall have sufficient tensile strength and water resistance to prevent irreparable damage or deterioration during transportation, storage at jobsite, and installation. The sheathing shall be continuous over the tendon length to be unbonded. The sheathing shall prevent the intrusion of cement paste and the escape of coating material. The sheathing may be a continuous tube or spiral wrapping.

15.2.6 *Grout*

15.2.6.1 Grout shall consist of a mixture of cement and water unless the gross inside cross-sectional area of the sheath exceeds four times the tendon cross-sectional area, in which case fine aggregate may be added.

15.2.6.2 Fly ash and pozzolanic material admixtures may be added at a rate not to exceed 30 lb per 94 lb of cement. The admixture shall conform to ASTM C 618.

15.2.6.3 An approved shrinkage compensating admixture shall be added to produce a maximum of 10 percent expansion by volume of the grout when measured unconfined.

15.2.6.4 Admixtures containing chlorides, fluorides or nitrates shall not be used. Other admixtures may be used provided approved tests or performance records show conclusively that they will have no harmful effects on the tendons, accessories or grout.

15.2.6.5 Fine aggregate shall conform to ASTM C 144, except that all material shall pass the No. 16 sieve.

15.2.6.6 Proportions of material shall be based on results of tests made on the grout before grouting is begun. The water content shall be the minimum necessary for proper placement and the water-cement ratio shall not exceed 0.50 by weight. The minimum 7-day compressive strength of 2-in. cubes molded, cured and tested in accordance with ASTM C 109 shall be 2500 psi.

15.3—Formwork

15.3.1—Formwork shall not restrain elastic shortening, deflection, or camber resulting from application of the prestressing force.

15.3.2—Form supports shall not be removed until sufficient prestressing force has been applied to support the dead load, formwork, and anticipated construction loads. When a structure is prestressed in two directions, formwork shall support the load which is redistributed by the partially completed stressing operation.

15.3.3—Formwork shall be sufficiently rigid to prevent displacement of the tendons beyond the tolerances of Section 15.4.

15.4—Placement and protection of tendons and accessories

15.4.1—Tendons and sheathing for use in bonded construction shall be free of grease, oil, paint, and other foreign matter. A light coat of rust is permissible, provided loose rust has been removed and the surface of the steel is not pitted.

15.4.2—Tendons for use in unbonded construction shall be clean and undamaged and shall be permanently protected as specified.

15.4.3—End anchorages which will be permanently protected with concrete shall be free of loose rust, grease, oil, and other foreign matter except paint.

15.4.4—Tendons, sheathing and anchorages shall be firmly supported to prevent displacement during concrete placement. They shall be placed with a tolerance of ±¼ in. in concrete dimensions of 8 in. or less, ±⅜ in. in concrete dimensions over 8 in. but not over 2 ft, and ±½ in. in concrete dimensions over 2 ft. These tolerances apply separately to both vertical and horizontal dimensions and may be different for each direction except that in slabs the horizontal tolerance shall not exceed 1 in. in 15 ft of tendon length.

15.4.5—Grout fittings and sheathing for bonded construction shall be adequately protected from collapse and other damage. Prior to placing concrete, the sheathing and grout fittings shall be examined for holes. All such holes shall be repaired. If the tendon is to remain ungrouted for more than 28 days from the time of tendon placement, temporary corrosion protection shall be provided.

15.4.6—The bearing surface between anchorages and concrete shall be concentric with and perpendicular to the tendons within ±1 deg.

15.5—Application of prestressing force

15.5.1—Tendons shall be stressed in the sequence, at the concrete strength, and at the construction stage indicated in the contract documents.

15.5.2—The prestressing force shall be determined by measuring tendon elongation and checking jack pressure with a calibrated gage or dynamometer. The gage or dynamometer shall have been calibrated within six months prior to use; any discrepancy which exceeds 5 percent shall be corrected. Elongation requirements shall be based on load-elongation curves for the steel used unless statistical data indicate that average values may be used. The contractor shall keep and submit a record for each tendon of the measured elongations and the gage pressures of dynamometer readings.

15.5.3—The total loss of prestress force in any tendon due to unreplaced broken elements shall not exceed 2 percent of the total prestress force.

15.6—Grouting

15.6.1—A dependable high pressure water supply of sufficient volume shall be provided before grouting is begun. Sheathing shall be freed of dirt and other foreign substances by thorough flushing with water immediately prior to grouting.

15.6.2—Grout shall be mixed in a high speed mechanical mixer and passed through a strainer into pumping equipment which has provision for recirculation. Pumping of grout shall begin as soon as possible after mixing and may be continued as long as the grout returns the proper consistency. Grout which has partially set shall be discarded.

15.6.3—Grout shall be injected into all voids between prestressing tendons, sheathing and anchorage fittings. Flow shall continue until grout of the consistency equivalent to that injected flows without the presence of air bubbles from vent openings. Vent openings shall be closed progressively in the direction of the flow. After all vent openings have been closed, the grouting pressure shall be raised to at least 50 psi and the injection hole plugged.

15.6.4—In the event of a blockage or an interruption of grouting, all grout shall be removed from the duct by flushing with water.

15.6.5—Provisions shall be made to keep tendons dry and keep water out of the conduit prior to grouting. The concrete around grouted tendons shall be maintained at a temperature of 45 F or higher for at least 3 days after grouting.

15.7—Shop drawings

Shop drawings for prestressed concrete shall be submitted for review and shall provide the following information in addition to that required by applicable provisions of Chapter 4, Formwork, and Chapter 5, Reinforcement.

15.7.1—The location of tendons throughout their length.

15.7.2—Size, details, location, materials, and stress grade (where applicable) for all tendons and accessories.

15.7.3—Jack clearances, jacking procedures, stressing sequence, initial tensioning forces, gage pressures, and tendon elongation.

15.7.4—Information required in Sections 15.2.1.2 and 15.2.1.3.

Notes

In the Sections of Chapter 15 listed below, specific acceptance is required:

15.2.1.2 and 15.2.1.3 Of the stress-strain curves for prestressing steel and other properties of non-ASTM material.

15.2.1.4 Of the stress loss in seating anchorage devices, friction coefficients and test data.

15.2.1.5 Of coatings to protect unbonded tendons from corrosion.

15.2.1.6 To perform burning or welding operations in the vicinity of tendons.

15.2.2.2 Of evidence of compliance with the dynamic test.

15.2.2.3 To use couplings at locations other than those indicated on the contract documents.

15.2.6.3 Of the shrinkage compensating admixture used in the grout.

15.2.6.4 Of tests or records showing no harmful effect of admixtures on tendons, accessories or grout.

15.7 Of shop drawings.

CHAPTER 16—TESTING

16.1—General

Concrete materials and operations will be tested and inspected as the work progresses. Failure to detect any defective work or material shall not in any way prevent later rejection when such defect is discovered nor shall it obligate the architect/engineer for final acceptance.

16.2—Testing agencies

16.2.1—The required testing services of Sections 16.3, 16.4, and 16.5 shall be performed by the testing agency designated in the contract documents. The services of Sections 16.3 and 16.4 will be performed at no cost to the contractor; those of Section 16.5 shall be paid for by the contractor.

16.2.2—The necessary testing services of Section 16.7.1 shall be performed by a testing agency acceptable to the architect/engineer at the contractor's expense.

16.2.3—All testing agencies shall meet the requirements of ASTM E 329.

16.2.4—Tests of concrete required in Section 16.3 shall be made by an ACI Concrete Field Testing Technician Grade 1, or equivalent. Equivalent certification programs shall include requirements for written and performance examinations as stipulated in ACI publication CP1.

16.3—Testing services

The following testing services shall be performed by the designated agency:

16.3.1—Review and/or check-test the contractor's proposed materials for compliance with the specifications.

16.3.2—Review and check-test the contractor's proposed mixture design when required by the architect/engineer.

16.3.3—Secure production samples of materials at plants or stockpiles during the course of the work and test for compliance with the specifications.

16.3.4—Conduct strength tests of the concrete during construction in accordance with the following procedures:

16.3.4.1 Secure composite samples in accordance with ASTM C 172. Each sample shall be obtained from a different batch of concrete on a random basis, avoiding any selection of the test batch other than by a number selected at random before commencement of concrete placement.

16.3.4.2 Mold and cure three specimens from each sample in accordance with ASTM C 31. Any deviations from the requirements of this Standard shall be recorded in the test report.

16.3.4.3 Test specimens in accordance with ASTM C 39. Two specimens shall be tested at 28 days for acceptance and one shall be tested at 7 days for information. The acceptance test results shall be the average of the strengths of the two specimens tested at 28 days. If one specimen in a test manifests evidence of improper sampling, molding or testing, it shall be discarded and the strength of the remaining cylinder shall be considered the test result. Should both specimens in a test show any of the above defects, the entire test shall be discarded. When high early strength concrete is used, the specimens shall be tested at the ages indicated in the contract documents.

16.3.4.4 Make at least one strength test for each 100 cu yd, or fraction thereof, of each mixture design of concrete placed in any 1 day. When the total quantity of concrete with a given mixture design is less than 50 cu yd, the strength tests may be waived by the architect/engineer if, in his judgment, adequate evidence of satisfactory strength is provided, such as strength test results for the same kind of concrete supplied on the same day and under comparable conditions to other work or other projects.

16.3.4.5 When accelerated testing of concrete is permitted as an alternative to standard testing, mold and cure two specimens from each sample in accordance with ASTM C 684, following the procedure specified by the architect/engineer. Make at least one accelerated strength test for each 100 yd^3 of concrete placed in any one day, of each mixture design of concrete placed in any one day, and one standard 28 day compressive strength test for at least every other accelerated strength test. Use these test results to maintain and update the correlation between accelerated and standard 28 day compressive strength tests.

16.3.4.6 Submit correlation data for the standard 28 day compressive strength test based on a minimum of 12 to 15 sets of test data covering the range of 2500 to 6000 psi concrete made with the same materials. Submit the correlation data for review.

16.3.5—Determine slump of the concrete sample for each strength test and whenever consistency of concrete appears to vary, using ASTM C 143.

16.3.6—Determine air content of normal weight concrete sample for each strength test in accordance with either ASTM C 231, ASTM C 173, or ASTM C 138.

16.3.7—Determine air content and unit weight of lightweight concrete sample for each strength test in accordance with ASTM C 173 and ASTM C 567.

16.3.8—Determine temperature of concrete sample for each strength test.

16.4—Additional services when required

The following services shall be performed by the designated agency when required by the architect/engineer:

16.4.1—Inspect concrete batching, mixing and delivery operations to the extent deemed necessary by the architect/engineer.

16.4.2—Sample concrete at point of placement and perform required tests.

16.4.3—Review the manufacturer's report for each shipment of cement, reinforcing steel and prestressing tendons and/or conduct laboratory tests or spot checks of the materials as received for compliance with specifications.

16.4.4—Other testing or inspection services as required.

16.5—Other services as needed

The following services shall be performed by the designated agency when necessary:

16.5.1—Additional testing and inspection required because of changes in materials or proportions requested by the contractor.

16.5.2—Additional testing of materials or concrete occasioned by their failure by test or inspection to meet specification requirements.

16.6—Duties and authorities of designated testing agency

16.6.1—Representatives of the agency shall inspect, sample and test the materials and the production of concrete as required by the architect/engineer. When it appears that any material furnished or work performed by the contractor fails to fulfill specification requirements, the testing agency shall report such deficiency to the architect/engineer and the contractor.

16.6.2—The agency shall report all test and inspection results to the architect/engineer, contractor, and concrete supplier immediately after the work is performed. All test reports shall include the exact location in the work at which the batch represented by a test was deposited. Reports of strength tests shall include detailed information on storage and curing of specimens prior to testing.

16.6.3—The testing agency and its representatives are not authorized to revoke, alter, relax, enlarge or release any requirement of the contract documents, nor to approve or accept any portion of the work.

16.7—Responsibilities and duties of contractor

16.7.1—The contractor shall provide the necessary testing services for the following:

16.7.1.1 Qualification of proposed materials and the establishment of mixture designs.

16.7.1.2 Other testing services needed or required by the contractor.

16.7.2—The use of testing services shall in no way relieve the contractor of the responsibility to furnish materials and construction in full compliance with the contract documents.

16.7.3—The contractor shall submit to the architect/

engineer the concrete materials and the mixture designs proposed for use with a written request for acceptance. This submittal shall include the results of all testing performed to qualify the materials and to establish the mixture designs. No concrete shall be placed in the work until the contractor has received such acceptance in writing.

16.7.4—To facilitate testing and inspection, the contractor shall:

16.7.4.1 Furnish any necessary labor to assist the designated testing agency in obtaining and handling samples at the project or other sources of materials.

16.7.4.2 Advise the designated testing agency sufficiently in advance of operations to allow for completion of quality tests and for the assignment of personnel.

16.7.4.3 Provide and maintain for the sole use of the testing agency adequate facilities for safe storage and proper curing of concrete test specimens on the project site for the first 24 hr as required by ASTM C 31.

16.7.4.4 Submit copies of mill test reports for shipments of cement, reinforcing steel and prestressing tendons to the architect/engineer when required.

Notes

In the Sections of Chapter 16 listed below, specific acceptance is required:

16.2.1 Of testing agency to perform services required in Section 16.7.1.

16.7.3 Of concrete materials and mixture designs.

CHAPTER 17—EVALUATION AND ACCEPTANCE OF CONCRETE STRENGTH

17.1—Evaluation of test results

17.1.1—Test results for standard molded and standard cured test cylinders shall be evaluated separately for each specified concrete mixture design. Such evaluation shall be valid only if tests have been conducted in accordance with procedures specified in Chapter 16.

17.1.2—For evaluation, each specified mixture design shall be represented by at least five tests.

17.2—Acceptance of concrete

The strength level of the concrete will be considered satisfactory so long as the averages of all sets of three consecutive strength results equal or exceed the specified strength f'_c, and no individual strength test result falls below the specified strength f'_c by more than 500 psi. These criteria apply also when accelerated strength testing is specified, unless another basis for acceptance is specified in the contract documents.

17.3—Testing of concrete in place

17.3.1—Testing by impact hammer, sonoscope, or other nondestructive device may be permitted by the architect/engineer to determine relative strengths at various locations in the structure as an aid in evaluating concrete strength in place or for selecting areas to be cored. Such tests, unless properly calibrated and correlated with other test data, shall not be used as a basis for acceptance or rejection.

17.3.2 *Core tests*

17.3.2.1 Where required, cores at least 2 in. in diameter shall be obtained and tested in accordance with ASTM C 42. If the concrete in the structure will be dry under service conditions, the cores shall be air dried (temperature 60 to 80 F, relative humidity less than 60 percent) for 7 days before testing and shall be tested dry. If the concrete in the structure will be more than superficially wet under service conditions, the cores shall be tested after moisture conditioning in accordance with ASTM C 42.

17.3.2.2 At least three representative cores shall be taken from each member or area of concrete in place that is considered potentially deficient. The location of cores shall be determined by the architect/engineer to least impair the strength of the structure. If, before testing, one or more of the cores shows evidence of having been damaged subsequent to or during removal from the structure, it shall be replaced with a new core.

17.3.2.3 Concrete in the area represented by a core test will be considered adequate if the average strength of the cores is equal to at least 85 percent of specified strength f'_c and if no single core is less than 75 percent of the specified strength f'_c.

17.3.2.4 Core holes shall be filled with low slump concrete or mortar. See Chapter 9, Repair of Surface Defects.

CHAPTER 18—ACCEPTANCE OF STRUCTURE

18.1—General

18.1.1—Completed concrete work which meets all applicable requirements will be accepted without qualification.

18.1.2—Completed concrete work which fails to meet one or more requirements but which has been repaired to bring it into compliance will be accepted without qualification.

18.1.3—Completed concrete work which fails to meet one or more requirements and which cannot be brought into compliance may be accepted or rejected as provided in these specifications or in the contract documents. In this event, modifications may be required to assure that remaining work complies with the requirements.

18.2—Dimensional tolerances

18.2.1—Formed surfaces resulting in concrete outlines smaller than permitted by the tolerances of Section 4.3.1 shall be considered potentially deficient in strength and subject to the provisions of Section 18.4.

18.2.2—Formed surfaces resulting in concrete outlines larger than permitted by the tolerances of Section 4.3.1 may be rejected and the excess material shall be subject to removal. If removal of the excess material is permitted, it shall be accomplished in such a manner as to maintain the strength of the section and to meet all other applicable requirements of function and appearance.

18.2.3—Concrete members cast in the wrong location may be rejected if the strength, appearance or function of the structure is adversely affected or misplaced items interfere with other construction.

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18.2.4—Inaccurately formed concrete surfaces exceeding the limits of Section 4.3.1 or of Section 13.3, and which are exposed to view, may be rejected and shall be repaired or removed and replaced if required.

18.2.5—Finished slabs exceeding the tolerances of Section 11.9 may be repaired provided that strength or appearance is not adversely affected. High spots may be removed with a terrazzo grinder, low spots filled with a patching compound, or other remedial measures performed as permitted.

18.3—Appearance

18.3.1—Architectural concrete with surface defects exceeding the limitations of Section 13.3.1 shall be removed and replaced.

18.3.2—Other concrete exposed to view with defects which adversely affect the appearance of the specified finish may be repaired only by acceptable methods.

18.3.3—Concrete not exposed to view is not subject to rejection for defective appearance.

18.4—Strength of structure

18.4.1—The strength of the structure in place will be considered potentially deficient if it fails to comply with any requirements which control the strength of the structure, including but not necessarily limited to the following conditions.

18.4.1.1 Low concrete strength as designated in Chapter 17.

18.4.1.2 Reinforcing steel size, quantity, strength, position, or arrangement at variance with the requirements of Chapter 5, Reinforcement, or the contract documents.

18.4.1.3 Concrete which differs from the required dimensions or location in such a manner as to reduce the strength.

18.4.1.4 Curing less than that specified.

18.4.1.5 Inadequate protection of concrete from extremes of temperature during early stages of hardening and strength development.

18.4.1.6 Mechanical injury as defined in Section 12.4, construction fires, accidents or premature removal of formwork likely to result in deficient strength.

18.4.1.7 Poor workmanship likely to result in deficient strength.

18.4.2—Structural analysis and/or additional testing may be required when the strength of the structure is considered potentially deficient.

18.4.3—Core tests in accordance with Section 17.3.2 may be required when the strength of the concrete in place is considered potentially deficient.

18.4.4—If core tests are inconclusive or impractical to obtain or if structural analysis does not confirm the safety of the structure, load tests may be required and their results evaluated in accordance with ACI 318.

18.4.5—Concrete work judged inadequate by structural analysis or by results of a load test shall be reinforced with additional construction if so directed by the architect/engineer, or shall be replaced, at the contractor's expense.

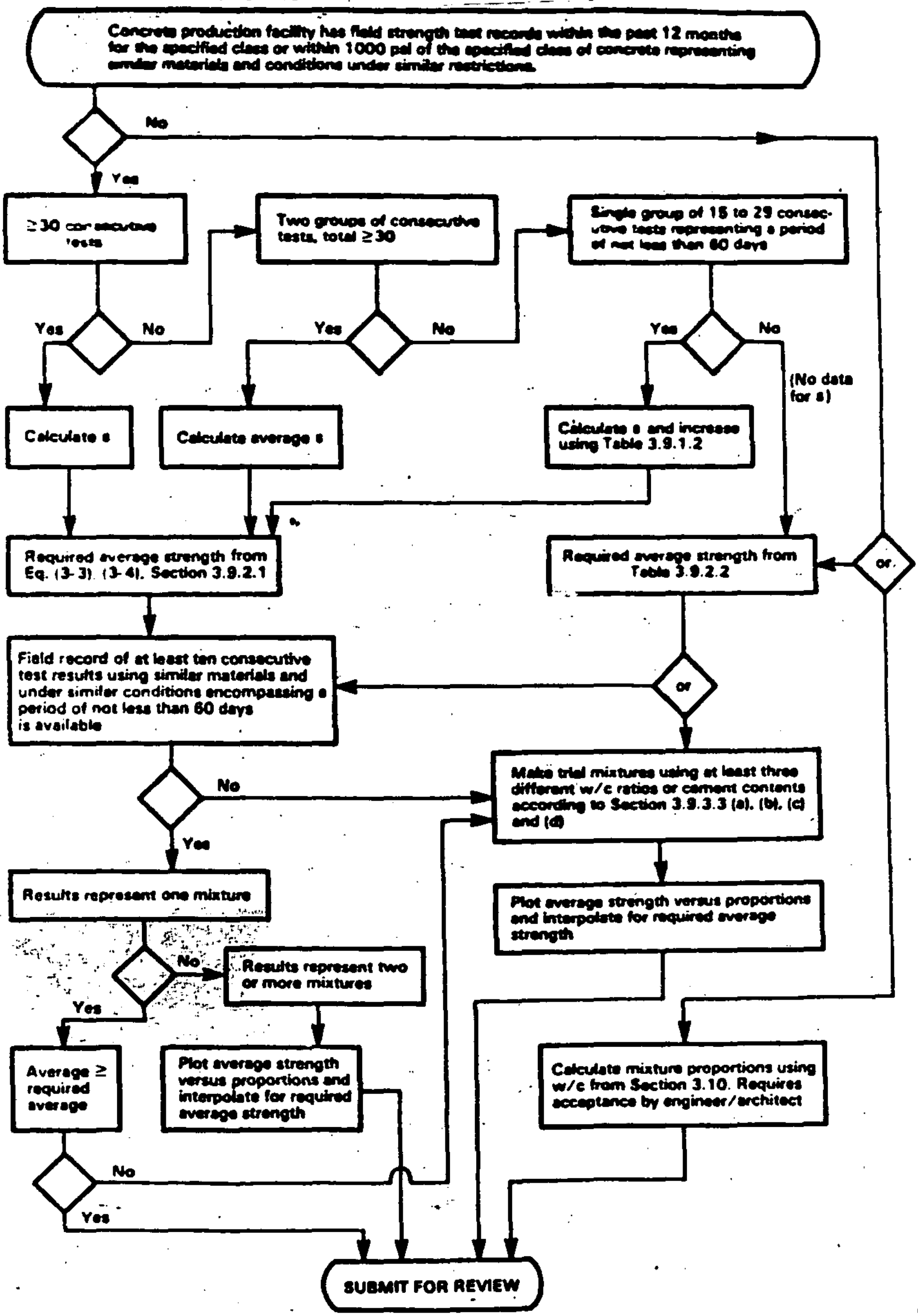
18.4.6—The contractor shall pay all costs incurred in providing the additional testing and/or analysis required by this chapter.

18.4.7—The owner will pay all costs of additional testing and/or analysis which is made at his request and which is not required by these specifications, or the contract documents.

Note

Permission is required if excess material is to be removed in accordance with Section 18.2.2.

APPENDIX A—FLOW CHART FOR SELECTION AND DOCUMENTATION OF CONCRETE PROPORTIONS



APPENDIX B—METRIC EQUIVALENTS

A1—General

This Specification has been presented using inch/pound units of measurement. The following list contains conversions from inch/pound units to International SI (Metric) units for all values which appear in the specification.

A2—Length

Inch/pound	SI (Metric)
[illegible] in.	9.5 mm
[illegible] in.	25.40 mm (exact)
[illegible] in.	25 mm
[illegible]	0.25 mm
[illegible] in.	0.76 mm
[illegible] in.	1.6 mm
[illegible] in.	3.2 mm
[illegible] in.	6.4 mm
[illegible] in.	13 mm
⅝ in.	16 mm
¾ in.	19 mm
1½ in.	38 mm
2 in.	51 mm
2½ in.	64 mm
3 in.	76 mm
4 in.	102 mm
5 in	127 mm
6 in.	152 mm
[illegible] in	203 mm
18 in.	457 mm
1 ft	0.305 m
2 ft	0.610 m
8 ft	2.44 m
[illegible] ft	3.05 m
[illegible] ft	30.48 m

A3—Area

Inch/pound	SI (Metric)
1 in.²	645 mm²
2 ft²	0.186 m²
100 ft²	9.29 m²
1000 ft²	92.9 m²

A4—Volume

Inch/pound	SI (Metric)
1 yd³	0.76 m³
50 yd³	38 m³
100 yd³	76.5 m³

A5—Weight, weight per unit of area

Inch/pound	SI (Metric)
2 lb	0.91 kg
25 lb per 100 ft²	1.2 kg/m²

A6—Density

Inch/Pound	SI (Metric)
115 lb per ft³	1840 kg/m³
135 lb per ft³	2160 kg/m³
160 lb per ft³	2560 kg/m³

A7—Stress

Inch/pound	SI (Metric)
300 psi	2.07 MPa
400 psi	2.76 MPa
500 psi	3.45 MPa
550 psi	3.79 MPa
600 psi	4.14 MPa
700 psi	4.83 MPa
900 psi	6.21 MPa
1200 psi	8.27 MPa
1800 psi	12.41 MPa
2500 psi	17.24 MPa
3000 psi	20.68 MPa
3500 psi	24.13 MPa
3750 psi	25.86 MPa
4000 psi	27.58 MPa
130,000 psi	896 MPa
140,000 psi	965 MPa
145,000 psi	1000 MPa
160,000 psi	1103 MPa

A8—Temperature

Fahrenheit	Celsius
0 F	−18 C
30 F	− 1 C
32 F	0 C
40 F	4 C
45 F	7 C
50 F	10 C
55 F	13 C
60 F	16 C
65 F	18 C
70 F	21 C
90 F	32 C
100 F	38 C
120 F	49 C

A9—Rate of temperature change

Fahrenheit	Celsius
3 F per unit of time	1.7 C/unit of time
5 F per unit of time	2.8 C/unit of time
30 F per unit of time	17 C/unit of time
50 F per unit of time	28 C/unit of time

A10—ASTM 11-81 sieve designations

Alternative	Standard
No. 16	1.18 mm
No. 30	600 μm

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- DETALLES Y DETALLADO DEL ACERO DE REFUERZO DEL CONCRETO (ACI 315-80)
- ADITIVOS PARA CONCRETO
- ADOQUINES DE CONCRETO
- AGREGADOS PARA CONCRETO
- DISEÑO Y CONSTRUCCIÓN DE ESTRUCTURAS DE BLOQUES DE CONCRETO
- CARTILLA DEL CONCRETO (ACI-SP1)
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- CIMBRAS DISEÑO
- CIMBRA: FALLAS, SEGURIDAD DE LA CIMBRA Y DESCIMBRADO. TOMO IV
- CIMBRAS Y MOLDES: GUÍA PRÁCTICA PARA SU CONSTRUCCIÓN Y USO
- PRÁCTICA RECOMENDABLE PARA LA MEDICIÓN, MEZCLADO, TRANSPORTE Y COLOCACIÓN DEL CONCRETO
- COMPACTACIÓN DEL CONCRETO
- ESTRUCTURAS DE CONCRETO PARA EL MEJORAMIENTO DEL MEDIO AMBIENTE
- CONCRETO ARQUITECTÓNICO COLADO EN OBRAS
- ESTRUCTURAS DE CONCRETO RESISTENTES AL FUEGO
- CONCRETO LANZADO
- EL CONCRETO EN LA OBRA. TOMO I
- EL CONCRETO EN LA OBRA. TOMO III
- INTRODUCCIÓN AL CONCRETO REFORZADO
- CONTROL DEL AGRIETAMIENTO DE ESTRUCTURAS DE CONCRETO
- CONTROL DE CALIDAD DEL CONCRETO
- CURADO ACELERADO DEL CONCRETO A PRESIÓN ATMOSFÉRICA
- CURADO DEL CONCRETO
- DEMOLICIÓN DE ESTRUCTURAS DE CONCRETO REFORZADO Y PRESFORZADO
- DISEÑO DE JUNTAS VIGA-COLUMNA EN ESTRUCTURAS DE CONCRETO ACI 352

colocación del concreto bajo temperatur extremas

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NORIEGA EDITORES

INSTITUTO MEXICANO DEL CEMENTO Y DEL CONCRETO, A.C.

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colocación del concreto bajo temperaturas extremas

contenido

PRIMERA PARTE

colocación del concreto en climas calurosos

COLOCACION DEL CONCRETO EN CLIMAS CALUROSOS

Informe del Comité ACI-305

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COLOCACION DEL CONCRETO EN CLIMAS CALUROSOS *
(ACI 305)

* Informe adoptado por el American Concrete Institute en agosto de 1977 para sustituir a la norma ACI 305-72.

sinopsis

El concreto mezclado, transportado y colocado bajo condiciones de alta temperatura, baja humedad o escaso viento requiere de una plena comprensión de los efectos que tales factores ambientales ejercen sobre las propiedades de éste y los sistemas de construcción.

Una vez que se han comprendido estos factores, se pueden tomar las medidas necesarias para eliminar o minimizar los efectos indeseables. Las dificultades más serias se presentan bajo condiciones climáticas y en tipos de construcción que son desusados para aquellos que efectúan el trabajo.

El presente informe del Comité define el término clima caluroso, enumera los posibles efectos desfavorables y propone prácticas con objeto de minimizarlas.

Entre dichas prácticas están medidas tan importantes como el enfriado previo de los ingredientes, considerar la temperatura del concreto al momento de la colocación, la longitud del acarreo, las instalaciones para el manejo del concreto en la obra y técnicas especiales para la dosificación, la colocación y el curado de éste. Se incluye una bien seleccionada bibliografía.

Palabras clave: acabados del concreto (concreto fresco); aditivos; agregados; **alta temperatura; colocación; concretos**; contenido de agua; contenido de cemento; construcción con concreto; **construcción en clima caluroso**; curado; enfriado; especificaciones; evaporación; fraguado (en-

durecimiento); hielo; humedad; inspección; laboratorios; métodos de producción; mezclado; preparación en la obra; presión del viento; proporcionamiento de mezclas; pruebas en el campo; resistencia a la compresión; retardantes; retemplado; sangrado (concreto); sub-base; temperatura; **transportación con bandas.**

CAPITULO 1

introducción

1.1 Aspectos generales

El clima caluroso provoca problemas en la fabricación, la colocación y el curado del concreto de cemento Portland, los cuales pueden afectar de manera adversa las propiedades y la durabilidad del concreto endurecido. Los objetivos de este informe son: identificar dichos problemas y explicar algunas prácticas para la colocación del concreto en clima caluroso con el fin de aliviar los efectos adversos que puedan presentarse en ausencia de dichas prácticas. La observación de éstas dará por resultado un concreto con mejores características, tanto al mezclarse, como en estado endurecido.

En otras publicaciones se podrá encontrar un estudio más profundo sobre temperaturas especiales, cambios de volumen y agrietamiento, problemas que se asocian con la masa y el concreto masivo.* Este informe presenta algunas sugerencias en relación con las preparaciones y los procedimientos que ayudan a reducir los riesgos de la colocación del concreto, en climas calurosos y en construcciones de tipo más general, tales como pavimentos, puentes, edificios y estructuras diversas.

Resulta importante reconocer que el daño que los climas calurosos ocasionan al concreto no pueden ser evitados por completo y, por lo tanto, será necesario contar con un criterio competente para seleccionar la

* ACI Committee 207, "Mass Concrete for Dams and Other Massive Structures", ACI Journal, *Proceedings*, V. 67, No. 4, abril 1970, pp. 273-309; y ACI Committee 224, "Control of Cracking in Concrete Structures", ACI Journal, *Proceedings*, V. 69, No. 12, diciembre 1972, Capitulo 7, pp. 736-745.

más apropiada relación entre calidad, economía y utilidad. Las precauciones que deben observarse o especificarse dependerán tanto del tipo de construcción y de la experiencia de la mano de obra local en su forma de desenvolverse en ese clima, como de las condiciones mismas del ambiente, la temperatura, la humedad relativa y la velocidad del viento.

Las dificultades más serias se presentan entre aquellas personas que se encuentran llevando a cabo el trabajo, sin estar habituadas al clima y a los diversos tipos de construcción. Puesto que sólo en raras ocasiones tienen éxito las improvisaciones de último momento, deben aplicarse medidas preventivas iniciales, haciendo hincapié en la evaluación de los materiales específicos, la planeación y las compras anticipadas, y la atinada coordinación de todas las etapas de la obra.

1.2 Definición de clima caluroso

Para propósitos de este informe, el clima caluroso se define como cualquier combinación de alta temperatura ambiente, baja humedad relativa y velocidad del viento, que tienda a perjudicar la calidad del concreto fresco o endurecido, o que de cualquiera otra manera provoque el desarrollo de anormalidades en las propiedades de éste. Los factores climáticos que afectan al concreto en climas calurosos son las altas temperaturas ambientales y la humedad relativa reducida, cuyos efectos pueden ser considerablemente más pronunciados con el incremento de la velocidad del viento. Los efectos del clima caluroso son más críticos durante los periodos de elevación de temperatura, de descenso de humedad relativa, o de ambas cosas. Pueden aparecer en cualquier época del año en climas calientes tropicales, o áridos, y, por lo general, se presentan durante la estación del verano en países de otras latitudes. Las medidas de precaución que se requieren en un día calmado y húmedo serán menos estrictas que las requeridas en un día seco y con viento, aun cuando la temperatura ambiente sea la misma.

1.3 Efectos del clima caluroso

1.3.1 Los efectos indeseables del clima caluroso en el concreto en estado plástico pueden incluir:

a) Incremento en los requerimientos de agua.
b) Incremento en la rapidez de la pérdida de revenimiento y la correspondiente tendencia a añadir agua en el lugar de la obra.
c) Incremento en la rapidez del fraguado, que tiene como resultado una mayor dificultad en el manejo, el acabado y el curado, y que aumenta la posibilidad de juntas frías.

d) Incremento en la tendencia al agrietamiento plástico.
e) Incremento en la dificultad para controlar el contenido de aire incluido.

1.3.2 Los efectos indeseables del clima caluroso en el concreto en estado endurecido pueden incluir:

a) Reducción de la resistencia como resultado del alto requerimiento de agua y de un incremento en el nivel de temperatura.
b) Incremento en la tendencia a la contracción por secado y al agrietamiento térmico diferencial.
c) Reducción de la durabilidad.
d) Reducción en la uniformidad de la apariencia superficial.

1.4 Efectos de factores adicionales

Existen otros factores que complican las operaciones en clima caluroso. Estos deben considerarse junto con los factores climáticos y pueden incluir:

a) El uso de cementos finamente molidos con rapidez de hidratación incrementada.
b) El uso de concreto con alta resistencia a la compresión, que requiere un más alto contenido de cemento.
c) El diseño de secciones delgadas de concreto, con el correspondiente aumento en el porcentaje de acero de refuerzo.
d) Mayor capacidad de los camiones para la entrega de concreto.
e) Requerimientos para la movilización de grandes volúmenes de concreto de bajo revenimiento a lo largo de mayores distancias, tanto horizontales como verticales.
f) Incremento en el uso de equipo para bombeo de concreto
g) Incremento en el uso de bandas transportadoras.
h) Necesidad, de índole económica, de continuar el trabajo en climas extremadamente calurosos.
i) El uso de cemento de contracción compensada.

CAPITULO 2

propiedades del concreto

2.1 Aspectos generales

2.1.1 Las propiedades del concreto, que hacen de él un excelente material de construcción, pueden verse afectadas de manera adversa por el clima caluroso, tal como se define en el Capitulo 1, pero dichos efectos perjudiciales pueden minimizarse por medio de los procedimientos prácticos de control descritos en este informe. La teoria básica relativa a las propiedades y al comportamiento del concreto se aplica, en términos generales, a concreto elaborado bajo condiciones óptimas. Los cambios de estación pueden tener como resultado ciertas condiciones que mucho distan de ser las ideales para la elaboración de concreto de la calidad deseada; por lo tanto, deben tomarse ciertas medidas, dentro del proceso de elaboración del concreto, con el fin de minimizar los efectos adversos. La resistencia, la impermeabilidad, la estabilidad dimensional y la resistencia a la acción atmosférica, el desgaste y el ataque químico, todos dependen del adecuado control de los materiales y del proporcionamiento de mezclas, de las temperaturas iniciales del concreto, y de las condiciones de temperatura y de humedad durante el periodo de la colocación y del curado. De antemano deben planearse las medidas necesarias con objeto de aliviar o eliminar los efectos adversos de las condiciones que privan en climas calurosos.

2.1.2 Los concretos mezclados, colocados y curados a elevadas temperaturas, normalmente desarrollan una resistencia inicial más alta que

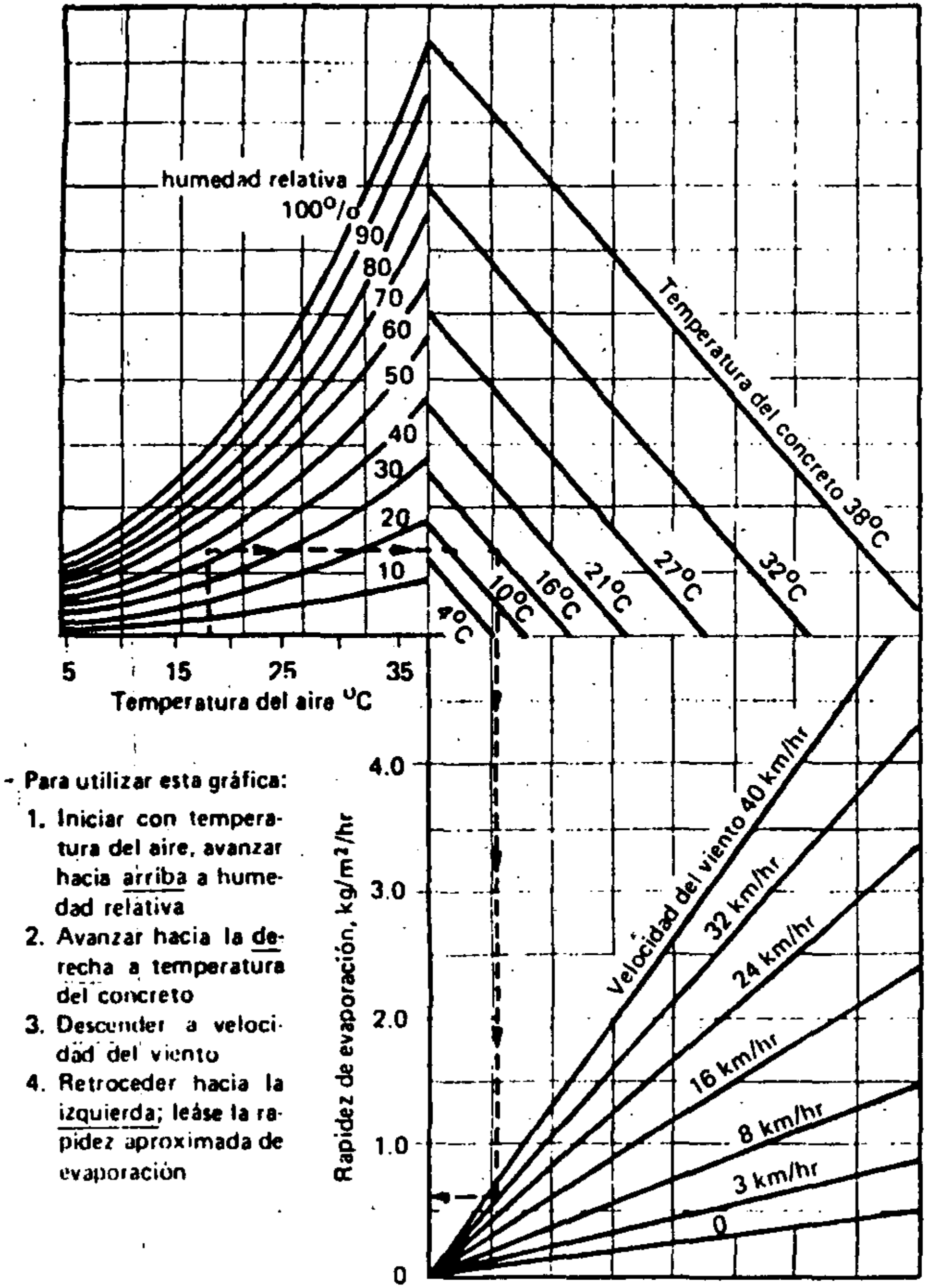


Fig. 2.1.5. Efecto de las temperaturas del concreto y el aire, la humedad relativa y la velocidad del viento sobre la rapidez de evaporación de la humedad de la superficie del concreto. Esta gráfica proporciona un método gráfico para calcular la pérdida de la humedad superficial bajo diversas condiciones atmosféricas. Para utilizar la gráfica, síganse los cuatro pasos arriba descritos. Si la rapidez de la evaporación se aproxima a 1.0 kg/m²/hr, es necesario tomar precauciones en

evaporación pueda aproximarse a 0.98 kg/m²/h. Estas medidas consisten en humedecer las sub-bases y las cimbras, colocar el concreto a la temperatura más baja posible, instalar rompevientos y sombrillas, reducir el tiempo entre la colocación del concreto y el inicio del curado, y minimizar la evaporación, en particular, durante las primeras horas siguientes a la colocación del concreto, empleando medidas adecuadas tales como la aplicación de humedad mediante el proceso de rociado. Las grietas ocasionadas por contracción plástica son difíciles de cerrar una vez que han aparecido. Dichas grietas pueden convertirse en puntos focales de otras formas de deterioro, puesto que permiten que la humedad y las sales disueltas penetren dentro del concreto y perjudiquen su comportamiento.

2.2 Efecto de la temperatura del concreto en el momento de su colocación

Conforme se permita la elevación de la temperatura del concreto en el momento de la colocación, se presentarán los siguientes efectos principales. Otros se describen en la Sección 1.3:

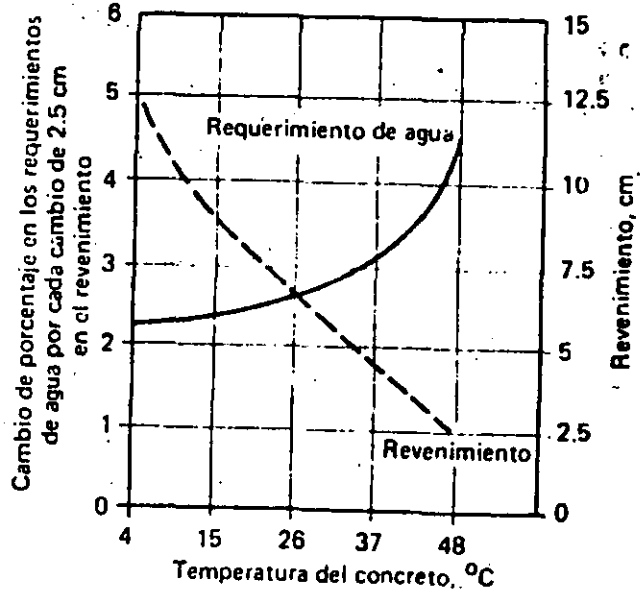


Fig. 2.3.3a. Efecto de la temperatura del concreto en el revenimiento y en el agua requerida con objeto de cambiar dicho revenimiento. Contenido de cemento: 307 kg/m³; porcentaje de aire: 4.5 ± 0.5; tamaño máximo del agregado, 38 mm (1 1/2 pulg); promedio de datos para cemen-

los producidos y curados a temperaturas normales, pero a los 28 días o después, la resistencia es, por lo general, más baja. Esto se ilustra en la Fig. 2.1.2.

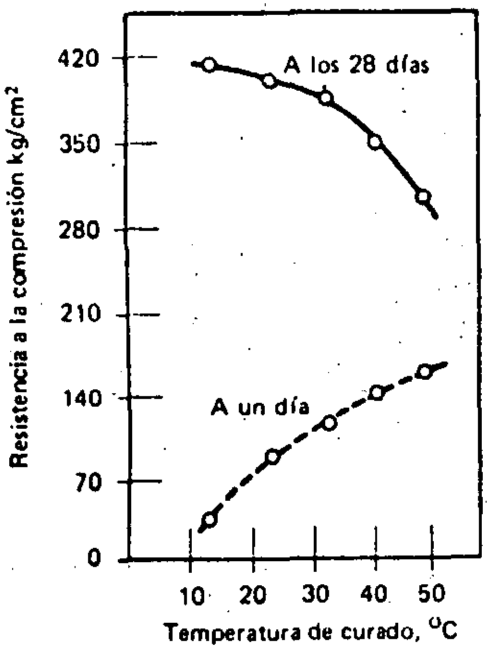


Fig. 2.1.2. La resistencia al primer día se incrementa con el aumento de la temperatura de curado, pero la resistencia a los 28 días disminuye con el aumento de la temperatura de curado. Referencia: "Structures and Physical Properties of Cement Pastes" (Verbeck and Helmuth, Simposio Japonés).

2.1.3 Las mezclas de prueba utilizadas para seleccionar las proporciones, usualmente se efectúan en laboratorios con temperaturas de alrededor de 23°C. Lo más probable es que la colocación del concreto se lleve a cabo, en climas calurosos, a temperaturas considerablemente más altas que la temperatura del laboratorio. Cuando exista un registro de comportamiento del concreto en la obra, en el cual se incluya el efecto de las temperaturas de temporada que pueden esperarse ,podrá utilizarse dicha información con el fin de establecer, o ajustar el proporcionamiento de mezclas para determinado proyecto, de acuerdo con el "Reglamento de las construcciones de concreto reforzado (ACI 318-71)", secciones 4.2.2. y 4.2.2.1.

2.1.4 Los investigadores han demostrado los efectos nocivos de la falta de curado y de las altas temperaturas en cilindros de prueba elaborados en los laboratorios. Se demostró que los cilindros moldeados y curados a temperaturas ambiente de 23°C, con 60% de humedad relativa, y a 38°C, con 25% de humedad relativa, producían una resistencia de sólo 73 y 62%, respectivamente, de la obtenida en cilindros estándar curados con humedad a 23°C durante 28 días. También se encontró que entre más largo fuera el tiempo transcurrido entre la elaboración y la colocación en almacenes húmedos estándar, mayor sería la reducción en la resistencia. Estas pruebas demuestran que un curado insuficiente, en especial en combinación con altas temperaturas de colocación, perjudica el proceso de hidratación y reduce la resistencia.

2.1.5 El agrietamiento por contracción plástica con frecuencia se asocia con la colocación de concreto a temperaturas calurosas en climas áridos y en cualquier momento puede suceder que la evaporación sea mayor que la rapidez con la cual el agua se eleva hasta la superficie del concreto recién colocado (sangrado). La alta temperatura del concreto, la alta temperatura ambiental, los fuertes vientos y la escasa humedad, o la combinación de estos factores, son causa de una rápida evaporación que, de manera significativa, incrementa la susceptibilidad de que se presente el agrietamiento por contracción plástica.

En climas húmedos, la alta temperatura del concreto es un factor mucho menos grave en la producción de contracción plástica. La Tabla 2.1.5 indica las temperaturas del concreto que pueden convertirse en críticas para la contracción plástica a diferentes humedades relativas. La rapidez de la evaporación puede calcularse utilizando la Figura 2.1.5. Deben tomarse medidas precautorias cuando se espere que la rapidez de

Tabla 2.1.5.— Temperaturas típicas del concreto para diversas humedades relativas potencialmente críticas en relación con el agrietamiento por contracción plástica *

Temperatura del concreto °C	Porcentaje de humedad relativa
41	90
38	80
35	70
32	60
29	50
27	40
24	30

* Temperatura máxima del concreto para diferentes humedades relativas, con objeto de limitar la rapidez de la evaporación hasta aproximadamente 1.0 kg/m²/hr, suponiendo una velocidad del viento de 16 km/hr y una diferencia de temperatura entre el concreto y el aire, de 6°C.

a) Se incrementará el contenido de agua para un revenimiento determinado. Figs. 2.3.3.a y 2.3.3b.

b) Este incremento en el contenido de agua provocará una disminución proporcionada de la resistencia y la durabilidad, y aumentará la contracción por secado. Fig. 2.3.4.

c) La pérdida de revenimiento se presentará más pronto después del mezclado y con una mayor rapidez. Estos dos son factores potenciales de dificultad en el manejo y la colocación del concreto. Figura 2.3.3a.

d) En climas áridos aumentará la probabilidad de que aparezcan grietas por contracción plástica. Fig. 2.1.5.

e) En secciones de grandes dimensiones existirá una mayor posibilidad de que se presenten diferencias entre la temperatura interior y la exterior, lo cual será suficiente para ocasionar un agrietamiento térmico.

f) El curado inicial rápido será cada vez más critico y su falta cada vez más perjudicial.

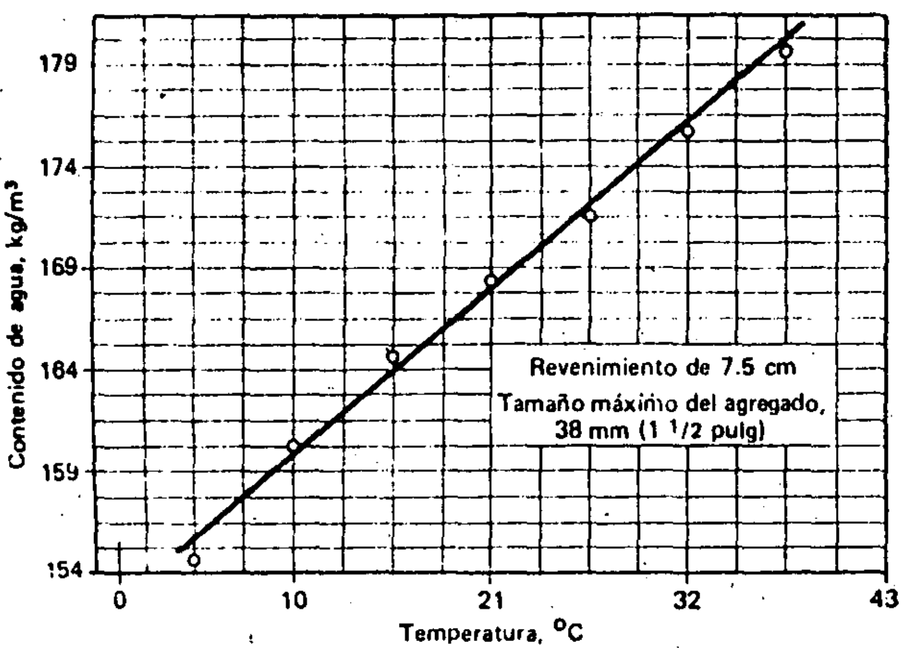


Fig. 2.3.3b. El requerimiento de agua de una mezcla de concreto aumenta con un incremento en la temperatura. Del Manual del Concreto del USBR, 8a. Edición, Fig. 118. También aparece como Fig. 1 en la Referencia 34.

2.2.2 En los tipos más comunes de construcción en clima caluroso, tal como se describen en la Sección 1.2, no resulta práctico recomendar una temperatura límite máxima, puesto que, dadas las circunstancias, la variación es muy amplia. Un límite que funcionaría a la perfección en un caso, sería altamente deficiente en otro. De acuerdo con esto, el Comité sólo puede hacer notar los efectos de temperaturas más elevadas en el concreto, tal como se menciona en las Secciones 1.3 y 2.2.1, y advertir que alguna temperatura, probablemente entre los 24 y 38°C, será el límite más favorable para obtener buenos resultados en cada una de las diversas operaciones efectuadas en clima caluroso y que deberá determinarse para cada obra. Las mezclas de prueba del concreto para dicha obra, deberán hacerse a la temperatura límite seleccionada y no a 21°C. Los procedimientos que describen las pruebas para las muestras de concreto, a temperaturas superiores a los 21°C se pueden encontar en la norma del ACI 223 "Práctica recomendada para el uso del concreto de contracción compensada".*

2.3 Efecto del agua

2.3.1 El agua, como ingrediente del concreto, influye en muchas de las más importantes propiedades de éste, tanto en su estado plástico como en el endurecido. Las altas temperaturas del agua producen más altas temperaturas en el concreto y, conforme aumenta la temperatura de éste, aumenta el requerimiento de agua y disminuye la resistencia en un concreto de consistencia similar. El agua adicional, sin hacer correcciones de su efecto en la relación agua/cemento, tendrá un efecto adverso en la calidad final del concreto colado en el lugar. La temperatura inicial del concreto, por sí misma, afecta la resistencia de éste tal como se describe en la Fig. 2.1.2.

2.3.2 Un contenido de agua más alto en una mezcla del concreto disminuye la resistencia, la durabilidad, la impermeabilidad y las propiedades relativas a éstas en el concreto resultante. Aunque adecuado a concretos colados en cualesquiera condiciones, esto apunta a la necesidad especial de controlar el uso del agua en el concreto colocado bajo condiciones de clima caluroso.

2.3.3 La Fig. 2.3.3a ilustra los efectos del aumento de temperatura del concreto en el revenimiento resultante de éste, cuando se mantiene constante la cantidad neta del agua de mezclado. Indica que puede esperarse que un aumento de 11°C en la temperatura, disminuya el reveni-

* Comité ACI 223, "Recommended Practice for the Use of Shrinkage-Compensating Concrete (ACI 223-77)", American Concrete Institute, Detroit, 1977, Sección 4.4, página 14.

miento en alrededor de 2.5 cm. La Figura 2.3.3a, también ilustra los cambios en el requerimiento de agua que pueden ser necesarios a fin de producir un cambio de 2.5 cm en el revenimiento, a diversos niveles de temperatura. En tanto que de los 4 a los 27°C solamente se requiere un cambio de agua de 2.25 a 2.5%, a los 49°C se requeriría un 4.5%. La figura 2.3.3b ilustra el efecto de la temperatura del concreto en el requerimiento de agua.

2.3.4 El resultado de una mayor exigencia de agua es un incremento en la contracción por secado. El enfriamiento que sigue a las altas temperaturas, a las cuales se efectúa el endurecimiento del concreto, aumenta la tendencia al agrietamiento de éste. La figura 2.3.4 muestra las magnitudes potenciales de la contracción por secado e ilustra la influencia del contenido de agua.

2.3.5 De todos los ingredientes del concreto, es el agua de mezclado la que tiene el mayor efecto, por unidad de peso, en la temperatura

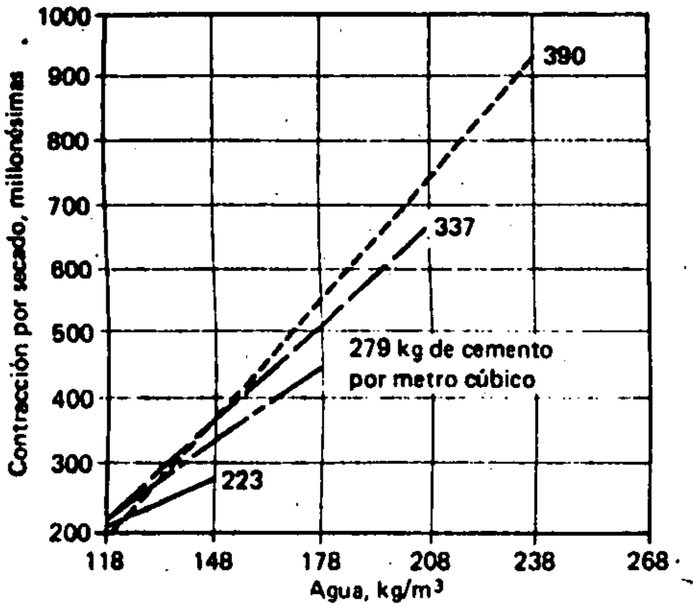


Fig. 2.3.4. Relación entre la contracción, el contenido de cemento y el contenido de agua. La gráfica indica que la contracción es una función directa del contenido unitario de agua en el concreto fresco. Nótese la poca influencia del contenido de agua en la contracción, sin tomar en consideración el contenido de cemento o la relación agua/cemento. El estrecho agrupamiento de estas curvas demuestra que la contracción en el momento del secado se rige principalmente por el contenido unitario de agua. Del Manual del Concreto del USBR, 8a. Edición, Fig. 8.

del concreto, ya que tiene un calor específico que equivale a 4 o 5 veces el del cemento o el del agregado. Es más fácil controlar la temperatura del agua que la del resto de los componentes y, aunque se emplea en cantidades más pequeñas que los demás, el uso de agua de mezclado fría tendrá un efecto reductor en la temperatura de colocación del concreto (véase la Fig. 2.3.5). Para una mezcla nominal de concreto que contenga 336 kg de cemento, 170 kg de agua, 1.850 kg de agregado/m³, un cambio de 2°C en la temperatura del agua provocará un cambio de 0.5°C en la temperatura del concreto. Por lo tanto, es aconsejable esforzarse por obtener agua fría y por mantenerla así mediante el aislamiento de tubos y tanques. Los tanques y camiones utilizados para almacenar o transportar agua deben ser térmicos y/o estar pintados de blanco. Se puede lograr una reducción en la temperatura del agua por medio de refrigeración mecánica, o mezclándola con hielo.

2.3.6 El uso de hielo como parte del agua de mezclado resulta altamente efectivo para reducir la temperatura del concreto, ya que sólo con

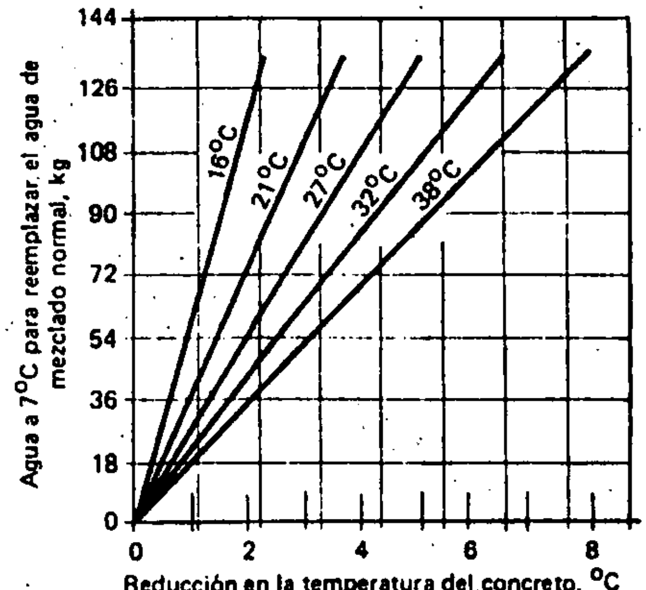


Fig. 2.3.5. Efecto del agua de mezclado enfriada sobre la temperatura del concreto. Las temperaturas son las normales del agua de mezclado. Estos valores se pueden aplicar a mezclas promedio hechas con agregados naturales típicos. La cantidad de agua enfriada no puede exceder de los requerimientos del agua de mezclado, la cual dependerá del contenido de humedad del agregado y de las proporciones de la mezcla. De la Referencia 20.

derretirse absorbe cálor a razón de 80 cal/g. Por lo tanto, para ser más efectivo, el hielo molido, triturado, astillado o raspado debe ser colocado directamente en la mezcladora para formar parte o constituir el volumen total del agua de mezclado. En la mezcla de concreto que se consideró en la Sección 2.3.5, la introducción de una cantidad de hielo equivalente al 50% del agua de mezclado, por peso, provocaría una disminución de la temperatura del concreto de 11°C solamente con derretirse, y el agua resultante a 0°C tendría un efecto enfriador adicional de 4°C. La figura 2.3.6 ilustra posibles reducciones en la temperatura del concreto mediante la sustitución del agua de mezclado a las temperaturas que se muestran, por diversas cantidades de hielo a 0°C. El mezclado debe continuar hasta que el hielo esté completamente derretido. El hielo triturado debe almacenarse a una temperatura tal que evite la formación de grumos mediante la recongelación de las partículas. Donde se requieren las cantidades máximas de hielo y/o de agua de mezclado enfriada, debe ponerse especial cuidado en el drenado de los agregados, con objeto de minimizar la humedad libre.

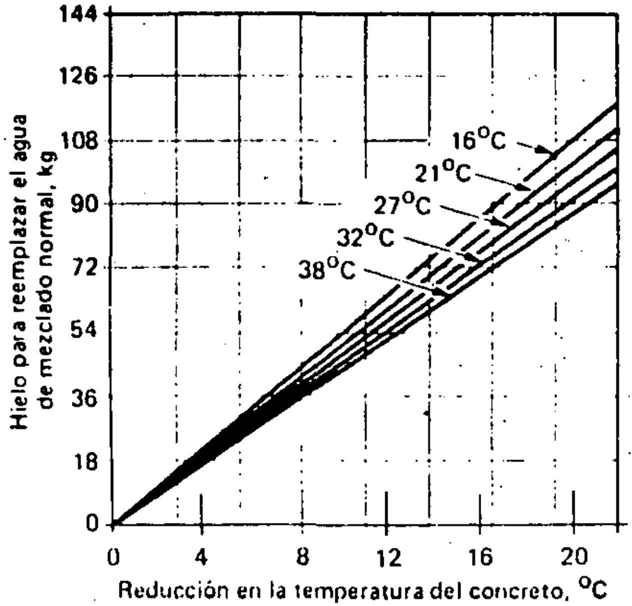


Fig. 2.3.6. Efecto del hielo en el agua de mezclado sobre la temperatura del concreto. Las temperaturas son las normales del agua de mezclado. Estos valores se pueden aplicar a mezclas promedio hechas con agregados naturales típicos. La cantidad de hielo añadida no puede exceder los requerimientos del agua de mezclado, la cual dependerá del contenido de humedad del agregado y de las proporciones de la mezcla. De la Referencia 20.

2.4 Efecto del cemento

2.4.1 El punto más importante a considerar, acerca del cemento como ingrediente del concreto en clima caluroso, es el efecto directo que la temperatura tiene en la rapidez con la que éste se hidrata. Las altas temperaturas del concreto aumentan la rapidez de hidratación y la rapidez del endurecimiento y, por lo general, tienen como resultado un incremento en el requerimiento de agua, contribuyendo, de este modo, a una reducción de la resistencia y a la contracción plástica. La Fig. 2.4.1 ilustra el efecto de la temperatura en el tiempo de fraguado de las mezclas de concreto, determinadas conforme a los procedimientos de la norma ASTM C 403 "Método estándar de prueba para determinar el tiempo de fraguado de mezclas de concreto por medio de la resistencia a la penetración". Aunque son limitados los datos en los cuales se basa la ilustración, sí indica un efecto decidido de la temperatura en el tiempo de fraguado, cuando las temperaturas de colocación del concreto están en el intervalo entre 10°C y 38°C. Asimismo, indica que la magnitud del efecto varía según la composición del cemento, cuando se utiliza un aditivo retardador de fraguado.

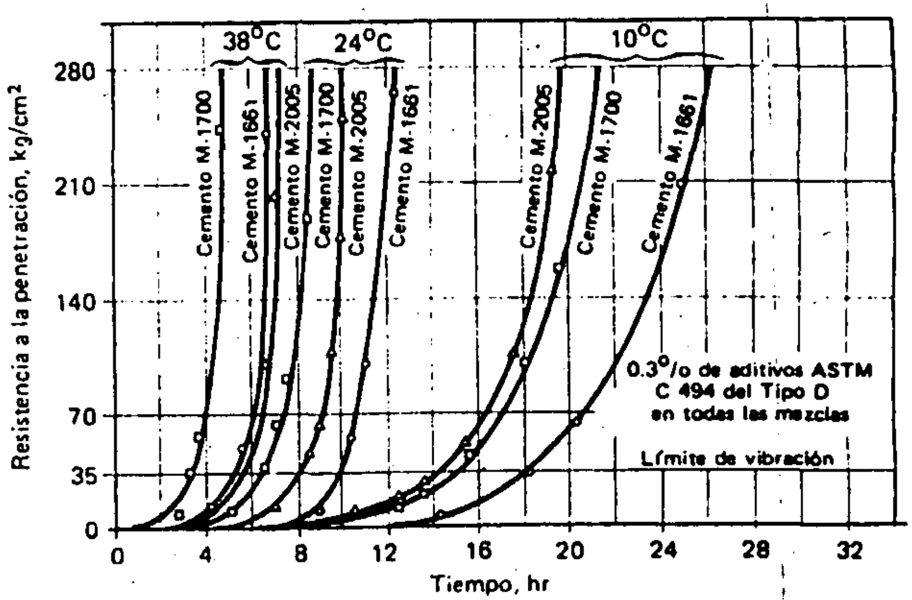


Fig. 2.4.1. La temperatura y la marca del cemento ejercen influencia sobre las características de endurecimiento de los morteros de concreto. De la Referencia 10.

2.4.2 El cemento caliente es el resultado del calor generado mecánicamente durante la molienda. La pérdida de calor durante el periodo de almacenamiento es lenta, por lo que el cemento puede entregarse a temperaturas relativamente altas. Aunque sólo del 10 al 15% del peso de una mezcla de concreto está constituido por cemento, su temperatura puede ser desde 0°C y llegar hasta los 55°C sobre la temperatura deseada para el concreto. Por lo tanto, resulta prudente establecer un límite máximo de 77°C para la temperatura del cemento al iniciar el mezclado. Si en el cemento existe una tendencia al fraguado falso, es factible que se acentúe la pérdida de revenimiento, en especial, en climas calurosos.

2.5 Efecto de los aditivos

2.5.1 Se ha encontrado que los aditivos que cumplen con las especificaciones de la norma ASTM C 494-71 "Especificación estándar para aditivos químicos para concreto" del Tipo B, retardadores, y del Tipo D, reductores de agua y retardadores, son beneficiosos para compensar algunas de las características indeseables del concreto colocado durante periodos con altas temperaturas ambientales. Estos aditivos afectan las siguientes propiedades de dicho concreto:

2.5.2 Los retardadores que se apegan a la norma ASTM C 494, del tipo B y del Tipo D, retrasan el tiempo de fraguado del concreto, según las medidas obtenidas en la prueba de penetración de la norma ASTM C 403 "Método estándar de prueba para determinar el fraguado de mezclas de concreto por medio de la resistencia a la penetración", pero no retardan la pérdida de revenimiento. Los aditivos del Tipo B y del Tipo D pueden utilizarse en proporciones variables, de manera que, conforme aumenta la temperatura, se pueden usar dosis más altas de dichos aditivos para lograr un tiempo de fraguado uniforme.

2.5.3 Conforme aumenta la temperatura del concreto aumentarán los requerimientos de agua para lograr la misma consistencia de éste durante la colocación. Los retardadores y reductores de agua disminuirán los requerimientos de ésta y, por lo tanto, en gran medida, reducirán este efecto de las altas temperaturas en el concreto.

2.5.4 Los aditivos retardadores y reductores de agua pueden incrementar un poco la rapidez de la pérdida de revenimiento. No obstante, por lo general se encontrará que aún después de que el revenimiento inicial ha aumentado lo suficiente para compensar cualquier aumento en la pérdida de éste, ocasionada por los aditivos, siguen siendo importantes la reducción del agua sobrante y los demás beneficios.

2.5.5 Los aditivos del tipo de ácido carboxílico hidroxilado y diversas variedades de otros tipos que llenan los requisitos del Tipo D, correspondientes a la norma ASTM C 494, aumentan el sangrado inicial del concreto sin aire incluido. Se ha visto que el sangrado inicial inducido, tal como el producido por estos tipos de aditivos, ha servido de ayuda para evitar que se seque la superficie superior del concreto colocado a alta temperatura ambiente y baja humedad. Sin embargo, si las condiciones de resequedad son tales que llegan a aparecer costras en la superficie, el subsecuente sangrado puede ocasionar descascaramiento. En estas condiciones debe utilizarse el rociado para prevenir dichas costras.

2.5.6 Los aditivos que cumplen con los requisitos de la norma ASTM C 494-71 "Especificación estándar para aditivos químicos para concreto", por lo general aumentarán la resistencia a la compresión y a la flexión del concreto. Con los ajustes apropiados en la mezcla, aquéllos se pueden utilizar para compensar la pérdida de resistencia provocada por la alta temperatura del concreto.

2.5.7 Aunque las pruebas cualitativas, tales como las de la ASTM, proporcionan un valioso procedimiento de evaluación para la selección de aditivos, cualquier uso sustancial de éstos en la continua producción de concreto debe estar precedido de pruebas que permitan la observación y la medición del comportamiento del producto, bajo las condiciones de operación de la planta de concreto, en combinación con los materiales para la fabricación de concreto que en ese momento se estén utilizando. Por lo general, la uniformidad de los resultados es tan importante, o más aún, que el resultado promedio en lo relacionado con cada una de las propiedades significativas del aditivo o del concreto.

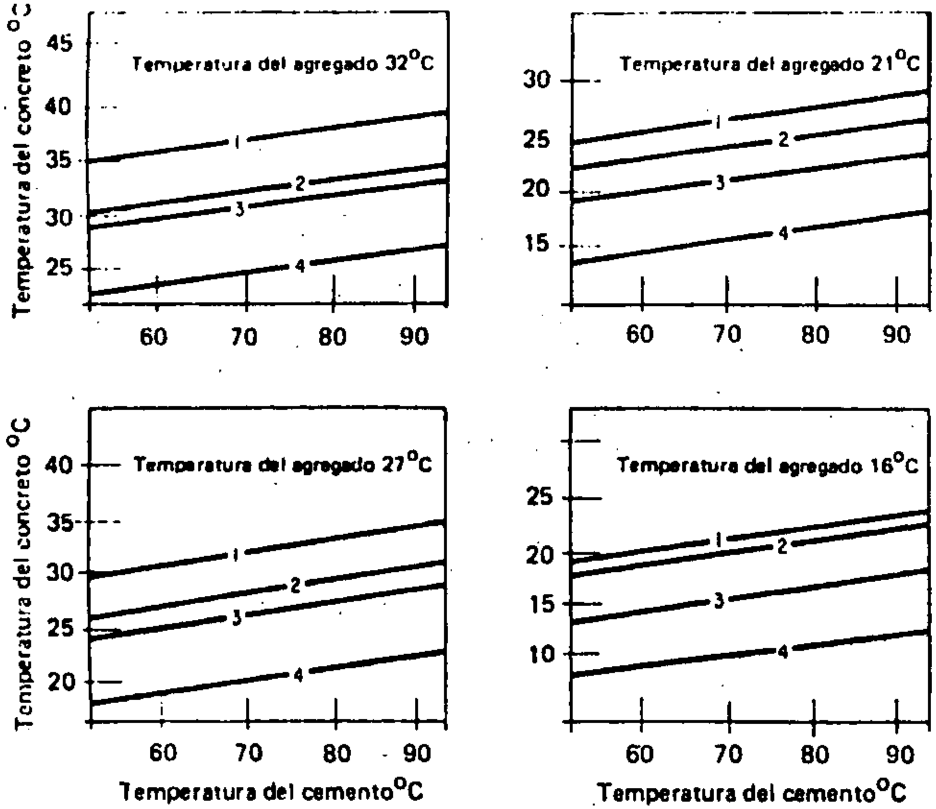
2.5.8 La evaluación del uso de un aditivo determinado o combinación de aditivos debe tener como base los resultados obtenidos con el concreto específico en cuestión y en las condiciones previstas en la obra, ya que los resultados obtenidos, en gran medida, están influenciados por las características del cemento y del agregado, y sus proporciones respectivas, al igual que por las prácticas de construcción y las condiciones ambientales. El operador de la planta, el contratista y el diseñador del proyecto de construcción están interesados en otros ángulos, ajenos a las propiedades del concreto, que se miden por medio de pruebas estándar. De primordial importancia pueden ser la trabajabilidad, el bombeo, la colocación del concreto y las características del acabado, el desarrollo temprano de la resistencia, la facilidad de volver a utilizar las cimbras y los moldes, la apariencia de las superficies descimbradas y demás. Son estos los puntos que pueden dictar la selección de un aditivo y la dosificación del mismo, aún más que las propiedades cubiertas por las especificaciones de costumbre.

CAPITULO 3

producción y entrega

3.1 Aspectos generales

Si los controles de proporcionamiento y de mezclado o los programas de entrega son tales, que el concreto llega al lugar de la colocación en malas condiciones, entonces no es posible adoptar las prácticas de colocación y de curado deseadas que se sugieren en el Capítulo 4. El control de la temperatura del concreto a través de la temperatura de los ingredientes, sólo se puede ejercer en el momento del mezclado. Utilizando las siguientes relaciones, se puede demostrar que para un concreto de proporciones convencionales, una reducción de 0.5°C en la temperatura de éste requiere una reducción de alrededor de 4°C en la temperatura del cemento, o de 2°C en la del agua, o de alrededor de 1°C en la del agregado. Puesto que la parte principal del concreto se compone de agregado, una reducción en la temperatura de éste provocará la mayor reducción en la temperatura del concreto (Fig. 3.1), por lo tanto deben emplearse todos los medios al alcance con el fin de mantener el agregado tan frío como sea posible. Esto se puede llevar a cabo, por ejemplo, manteniendo todos los componentes a la sombra. La evaporación y el enfriamiento directo que se obtienen por medio del rociado a mano o con aparatos de los cúmulos de agregado grueso, es un recurso efectivo para reducir las temperaturas del agregado. Este rociado no se puede efectuar de manera fortuita, ya que esto puede conducir a una excesiva variación en la humedad de la superficie y, por lo tanto, perjudicar la



Curva 1 — Agua de mezclado a la temperatura del agregado
Curva 2 — Agua de mezclado a 10°C
Curva 3 — Agua de mezclado a la temperatura del agregado; 25°/o del agua de mezclado por peso, reemplazada por hielo
Curva 4 - Agua de mezclado a la temperatura del agregado; 50°/o del agua de mezclado por peso, reemplazada por hielo.

Fig. 3.1. Influencia de la temperatura de los ingredientes del concreto en la temperatura del mismo. Cálculo obtenido de las ecuaciones de la Sección 3.1.

uniformidad del revenimiento. No obstante, los beneficios son muchos y compensan una cuidadosa atención. Las ecuaciones para el cálculo de las temperaturas del concreto recién mezclado son las siguientes:

Sin hielo [Unidades del mks y del Sistema Internacional (SI)]

$$T = \frac{0.22\,(Ta\;Wa + Tc\;Wc) + Tw\;Ww + Twa\;Wwa}{0.22\,(Wa + Wc) + Ww + Wwa}$$

Con hielo [Unidades del mks y del Sistema Internacional (SI)]

$$T = \frac{0.22\,(Ta\;Wa + Tc\;Wc)}{0.22\,(Wa + Wc) + Ww + Wi + Wwa} + \frac{(Ww - Wi)\,Tw + Wwa\;Ta - 79.6Wi}{0.22\,(Wa + Wc) + Ww + Wi + Wwa}$$

donde:

T = Temperatura del concreto recién mezclado, grados C.

Ta, Tc, Tw = Temperatura del agregado, del cemento y del agua de mezclado, respectivamente, grados C.

Wa, Wc, Ww, Wwa, Wi = Peso del agregado, del cemento, del agua de mezclado, del agua libre en el agregado y del hielo, respectivamente, en kg.

3.2 Agua

El enfriado del agua de mezclado y/o el uso de hielo es importante con motivo de las relaciones que se dan en el Capítulo 2. La temperatura del agua se baja con facilidad, y aunque el agua se utiliza en cantidades más pequeñas que el resto de los ingredientes, el uso de agua de mezclado fría tendrá como resultado una moderada reducción en la temperatura de colocación del concreto. El uso de hielo, aunque para esto sea necesario contar con equipo para fabricarlo o para triturarlo, además de una muy exacta dosificación, resulta mucho más efectivo que el del agua fría.

3.3 Dosificación y mezclado

La dosificación, el mezclado y la transportación del concreto se describe en los Capítulos 4 y 5 de la Norma ACI 304-73.*

3.3.1 Los procedimientos de dosificación tienen efectos importantes en la facilidad de producir concreto uniforme perfectamente mezclado, tanto en mezcladoras estacionarias, como en camiones mezcladoras. Por lo general se acrecienta la uniformidad en el mezclado cargando simultáneamente, mediante una banda transportadora, el agregado, el cemento y el agua. No obstante, cuando la planta mezcladora está localizada a

* "Práctica recomendable para la medición, mezclado, transporte y colocacion del concreto". Instituto Mexicano del Cemento y del Concreto, A. C. México, …

cierta distancia de la obra y la temperatura ambiente, la longitud de acarreo y otros factores, son de naturaleza tal que provocan dificultades con la pérdida de revenimiento, el alto requerimiento de agua o el endurecimiento rápido, deben tomarse en consideración los procedimientos mediante los cuales se permita el mezclado después de haber llegado al lugar de la obra y así evitar, o minimizar, el contacto entre el cemento y el agregado húmedo antes del mezclado. En obras sumamente grandes, el volumen del concreto puede ser el suficiente como para justificar la instalación de una planta completa en el lugar de la obra.

3.3.1.1 En obras de mediana extensión puede ser factible colocar instalaciones para la dosificación del concreto en el lugar mismo de la obra, o cerca de él. En este caso, los agregados y no más de alrededor del 80% del agua de mezclado pueden incorporarse, en algunas instalaciones a distancia regular, añadiendo el cemento y el resto del agua en el lugar de la obra.

3.3.1.2 En una obra pequeña donde no son factibles ni una planta completa ni una dosificación de cemento, se pueden emplear procedimientos especiales de dosificación para minimizar el contacto significativo entre el cemento y el agua o el agregado húmedo. Cuando el cemento y los agregados se cargan por medio de una banda, prácticamente todo el cemento se humedece y son pocas las ventajas de retrasar el mezclado hasta llegar al lugar de la obra. Si el cemento es el último ingrediente en añadirse, pero la olla de la mezcladora se está moviendo a una velocidad normal de 10 a 18 rpm, los resultados serán similares a los alcanzados en la dosificación por medio de una banda. Si durante el periodo de carga, la velocidad de la mezcladora se reduce hasta llegar a ser de 4 a 6 rpm, gran parte del cemento se humedece y los resultados son ligeramente mejores que los del caso anterior. No obstante, será muy poco el cemento humedecido, por contacto con el agregado húmedo si se detiene por completo el movimiento de la mezcladora, en tanto se añade el cemento y no se echa a andar sino hasta que se efectúa el mezclado en el lugar de la obra. Este procedimiento permitía retrasos de 16 hrs o más en el mezclado, en los días en que eran comunes las mezcladoras de eje horizontal cargadas a través de una escotilla. Los modernos camiones mezcladoras de eje inclinado sólo en raras ocasiones están provistos de escotillas semejantes, pero cuando lo están, es posible extender sustancialmente el tiempo de entrega si se utiliza este procedimiento. Sin embargo, aun con algunas limitaciones, es posible cargar el cemento como último ingrediente en una olla sin movimiento. Si se utiliza este método, la pérdida de resistencia por retrasos de hasta 3 hrs en el mezclado después de la carga, será de menos del 5 o 10%. Para evitar la pérdida de una cantidad excesiva de cemento seco, será necesario reducir el tamaño de la carga entre un 10 y un 20%. El uso de este procedimiento aumenta de manera importante el tiempo requerido para cargar un camión y reduce, con mucho, la capacidad de la planta.

3.3.2 La cantidad de mezclado y de agitación deben mantenerse al nivel más bajo posible. El número de revoluciones para la velocidad de mezclado debe mantenerse, por lo general, a 70, 100, o cuando más a 125 rpm. Con el concreto mezclado en la planta, o en el camión mezcladora en la planta, debe tomarse en consideración la conveniencia de transportar el concreto con la olla sin movimiento en el transcurso del recorrido. Esto requeriá un pequeño remezclado en la obra antes de descargar. En terreno accidentado puede ser necesario agitar el concreto con alto revenimiento para evitar derrames.

3.3.3 Para minimizar el calor producido por los rayos del sol, resulta útil pintar de blanco las superficies de la mezcladora. Tomando como base una hora de tiempo de entrega en los meses de verano, el concreto en una olla blanca y limpia debe estar 1.4°C más fresco que en una olla roja y 0.3°C más fresco que en una de color crema. Si una olla vacía permanece al sol durante un periodo de tiempo más o menos extenso antes de la dosificación del concreto, el calor almacenado en la olla metálica producirá temperaturas de 0.3 a 0.5°C menores en una olla blanca que las que se producirían en una roja o amarilla. Se ha sugerido rociar la olla con agua antes de la dosificación o durante el transporte como medida para reducir la temperatura del concreto, pero sólo se obtendrán beneficios marginales con este procedimiento.

2.3.4 En zonas de baja humedad relativa se ha sugerido que el concreto se puede enfriar mediante un rociado continuo de la olla con agua para obtener un enfriado por evaporación. No se sabe que existan verdaderos datos de campo a este respecto, pero según cálculos basados en la Fig. 2.1.5, se deduce que los efectos del enfriado serían muy pocos. En condiciones ambientales que produjeran una velocidad de evaporación de 4 kg/m²/h, la velocidad de enfriado sería de alrededor de 2°C por hora. El enfriado obtenido a una velocidad de evaporación menor sería también proporcionalmente menor.

3.4 Entrega

La hidratación del cemento, la elevación de la temperatura, la pérdida de revenimiento, el desgaste del agregado y la pérdida de aire aumentan con el paso del tiempo; por lo tanto, debe mantenerse a un mínimo absoluto el lapso de tiempo entre el mezclado y la entrega. Esto presenta algunos problemas especiales en las operaciones de concreto premezclado. Debe prestarse especial atención a la coordinación de la salida de los camiones con la rapidez de colocación, a fin de evitar restrasos en la entrega. Cuando el periodo transcurrido entre la dosificación y la colocación es tan largo como para tener por resultado incrementos importantes en los requerimientos de agua de mezclado, o en la pérdida de revenimiento, debe retrasarse el mezclado en los camiones hasta que sólo quede el tiempo suficiente para terminar este proceso antes de la colocación del concreto.

3.5 Retemplado

Tanto la investigación en laboratorio como la experiencia en el campo sugieren que la reducción en la resistencia, al igual que otros efectos perjudiciales del clima caluroso, sean directamente proporcionales a la cantidad de agua de retemplado añadida. No debe permitirse la adición de agua al llegar a la obra, con excepción de aquella requerida inicialmente para ajustarse al revenimiento especificado, siempre que dicha adición no exceda los límites de la máxima relación agua/cemento especificada. Debe prohibirse cualquier adición de agua posterior.

3.6 Control

Para minimizar los efectos adversos que se describen en la Sección 2.2 se utilizan varios métodos de control para el mezclado y la entrega en clima caluroso. Las buenas prácticas acostumbradas en la colocación del concreto para minimizar los tiempos de mezclado y de entrega, y el contenido de agua, requieren de mayor diligencia y coordinación durante el clima caluroso.

CAPITULO 4

colocación y curado

4.1 Aspectos generales

4.1.1 En muchos aspectos, los requisitos para obtener buenos resultados durante la colocación y curado del concreto en climas calurosos no suelen ser distintos de los necesarios en otras estaciones. Existen las mismas necesidades: a) que el concreto se maneje y transporte con un mínimo de segregación y de pérdida de revenimiento; b) que el concreto se coloque en el lugar donde va a permanecer; c) que el concreto se cuele en capas lo suficientemente delgadas a fin de asegurar el vibrado hasta la parte profunda de la capa inmediatamente inferior; d) que las juntas se fabriquen en concreto sólido y limpio; e) que las operaciones de acabado y su ritmo se guíen únicamente por el grado de terminación del concreto respecto a ellas y nada más; f) que el curado se conduzca de manera tal que en ningún momento durante el periodo descrito, carezca el concreto de la humedad y control de temperatura necesarios, de manera que la hidratación continúe desarrollando la totalidad del potencial de resistencia y durabilidad del concreto.

4.1.2 En otras normas y referencias del ACI se encuentran disponibles los detalles de los procedimientos requeridos en todas las estaciones y otros requisitos estándar. El presente capítulo tiene como propósito destacar los factores peculiares del clima caluroso que pueden afectar a estas operaciones y al concreto resultante, y recomendar aquellas que deben observ[illegible] co[illegible]to [illegible]eve[illegible] ha[illegible]sal[illegible] in[illegible]ia.

4.2 Preparativos para la colocación y el curado

4.2.1 Los preparativos para la colocación y el curado en clima caluroso incluyen el reconocimiento, al iniciar el trabajo, de que se presentarán ciertas condiciones anormales, las cuales requerirán de algunos detalles de preparación que no se pueden proporcionar eficazmente en el preciso momento anterior a la colocación del concreto. Si se espera que las temperaturas del concreto en el momento de la colocación de éste sean anormalmente altas, deben llevarse a cabo preparativos para que el transporte, la colocación, la compactación y el acabado de dicho concreto se efectúen con la mayor rapidez posible.

4.2.1.1 Esto significa, en primer lugar, que la entrega del concreto en la obra debe estar programada de manera que se coloque inmediatamente al llegar, en particular, la primera carga. Muchas operaciones de colocación de concreto tienen un mal principio porque la entrega de éste se hizo cuando aún no estaba lista para recibirlo la zona de trabajo, y el control del revenimiento se perdió en este momento tan crítico.

4.2.1.2 El equipo para la colocación del concreto debe contar con la capacidad adecuada para cumplir con sus funciones de manera eficiente, con el fin de no provocar retrasos en etapas distantes del trabajo. Debe haber el suficiente equipo de vibración y la mano de obra necesaria para la compactación inmediata del concreto después de su colocación y para mantener el ritmo de ésta en áreas difíciles. Todo el equipo debe estar en condiciones óptimas de operación. Las descomposturas, o los retrasos que detienen o hacen más lenta la colocación, pueden afectar seriamente la calidad del trabajo. Las juntas frías pueden quedar en evidencia cuando se retire la cimbra; una falla en la vibración puede causar una obvia falta de compactación.

4.2.1.3 Debido a la mayor rapidez de pérdida de revenimiento en clima caluroso, el esfuerzo en el equipo de vibración será mayor. De acuerdo con esto deben tenerse varios vibradores de repuesto, por lo menos uno por cada tres vibradores en uso. Una operación de colocación de concreto se va a ver en dificultades, en especial en clima caluroso, si el equipo de vibración falla y el equipo de repuesto no es el adecuado. De ser posible, deben efectuarse arreglos previos con el fin de poder contar con otra grúa, o bomba adicional, a la mayor brevedad en caso de una descompostura del equipo.

4.2.2 Cuando se van a construir losas planas a nivel del terreno, sobre una base, una planeación previa puede permitir la programación de las losas una vez que se hayan levantado los muros, o que se haya colocado el techo, y de esta manera proporcionar una protección contra el viento, o una sombra, o ambos. Si prevalece un viento seco, podría valer la pena planear una protección temporal contra el viento. En cualquier caso, la obra debe contar con una amplia provisión de agua, mangueras y boquillas pulverizadoras. La sub-base debe estar húmeda pero sin agua estancada, ni puntos suaves en el momento de la colocación del concreto. La pulverización se puede utilizar con el fin de refrescar y humedecer el aire circundante y, de este modo, evitar una excesiva evaporación de los elementos planos durante el acabado. Las boquillas pulverizadoras que se empleen deben producir una capa de rocío semejante al que deja la neblina y no debe confundírseles con boquillas de manguera de jardín, las cuales producen un rocío excesivamente grueso.

4.2.3 Los preparativos para la colocación del concreto incluyen la localización apropiada y la preparación de las juntas de construcción. Debido a que son más rápidos el fraguado y el endurecimiento del concreto, el ritmo de limpieza por medio de varios métodos, como son el labrado en estado fresco, o la aplicación de retardantes en la superficie, se vuelve más crítico en clima caluroso. Deben hacerse preparativos con el fin de proporcionar una rápida y adecuada atención a estos renglones, en el momento indicado.

4.2.4 Los planes para el trabajo deben incluir preparativos para limitar la temperatura del concreto en el momento de su colocación, de acuerdo con una meticulosa consideración de su potencial y sus efectos, tal como se explicó en las Secciones 1.3, 2.2.1 y 2.2.2, ya que éstas se aplican al carácter privativo de las condiciones prevalecientes en la obra. Conforme se alcanza y se sobrepasa la temperatura límite seleccionada, que es, por lo general, aunque no siempre, entre 24 y 38°C, es todavía más factible que se presenten los efectos poco favorables de la alta temperatura.

4.2.4.1 Sin importar cual límite de temperatura haya sido considerado el adecuado, siempre resultará más fácil mantenerlo si las mezcladoras, las bandas, la tubería de bombeo y las canaletas se mantienen a la sombra. Donde esto no sea posible, este equipo absorberá considerablemente menos calor si se pinta y se mantiene pintado de blanco. Las tuberías de bombeo se pueden conservar mucho más frescas si se las cubre con tela de yute mojada, que se mantendrá así por medio de una manguera constantemente abierta. Cuando la temperatura durante el día y las condiciones de resequedad sean críticas, programar el inicio de la colocación del concreto en las últimas horas de la tarde mejorará notablemente las condiciones para dicha colocación. Se ha encontrado que para losas masivas y pavimentos, este procedimiento ha tenido como resultado una contracción térmica y un agrietamiento considerablemente menores. El concreto que se coloca en las primeras horas de la mañana puede alcanzar una muy alta temperatura que resulta definitivamente indeseable, en particular al mediodía, que es cuando se presentan la máxima irradiación de sol y calor de hidratación. Al enfriarse, este concreto se podría ver expuesto a un severo esfuerzo térmico.

4.2.5 Finalmente, vemos que los preparativos para la colocación de concreto en clima caluroso incluyen medidas especiales necesarias para el curado y la protección apropiada de éste, puesto que el clima caluroso provoca un rápido secado. Si se desea evitar el agrietamiento y el daño grave ocasionado por el secado, se debe contar con instalaciones listas para proteger, con rapidez, todas las superficies expuestas. Debe dársele preferencia al curado húmedo en la mayoría de las obras de concreto, pero se acepta que una pronta aplicación del compuesto de curado con pigmento blanco (ASTM C 309, "Standard Specifications for Liquid Membrane-Forming Compounds for Curing Concrete") del Tipo 2, resulta más práctico para el curado de grandes áreas de losas planas a nivel del terreno sobre sub-base, como son la pavimentación de carreteras y el recubrimiento de canales. En la Norma del Comité ACI 308 sobre curados,* se describen otras alternativas para el curado. El curado con agua debe ser continuo y esta continuidad se asegura, con más facilidad, si se toman medidas a fin de cubrir todas las superficies expuestas, verticales, horizontales o de cualquier otra forma, con material saturado (yute, mantas de algodón, alfombras viejas, etc.) que se mantiene mojado por medio de una manguera constantemente abierta. Este material debe mantenerse en contacto directo con la superficie de concreto en todo momento. Los ciclos alternos de mojado y secado propician el desarrollo de un patrón de agrietamiento y deben evitarse. El agua de curado no debe ser más fría que el concreto a causa de los esfuerzos por cambios de temperatura, que podrían presentarse con el consiguiente agrietamiento.

4.3 Colocación y acabado

4.3.1 La prontitud en la colocación del concreto y su acabado reducen considerablemente las dificultades del clima caluroso. Los retrasos aumentan la pérdida de revenimiento, e invitan a la adición de agua con el fin de contrarrestarla. Todas y cada una de las operaciones de acabado del concreto deben efectuarse con prontitud, cuando el concreto está listo para ello. Es necesario asegurarse de que el concreto no se coloque en las cimbras en un tiempo menor que el que toma a los hombres y equipo, compactarlo, o bien que los trabajadores le den el acabado apropiado a mano. Si el ritmo de colocación no está en coordinación con los trabajadores y con el equipo disponible, la obra rápidamente se verá marcada por juntas frías, una compactación deficiente y acabados irregulares de la superficie.

4.3.2 Sin tomar en consideración el espesor de las capas de concreto, cuando éste se coloca bajo temperaturas normales, cada capa tal

* Comité ACI 308 "Práctica recomendable para el curado del concreto", Instituto [illegible] [illegible] del [illegible], [illegible] Mé[illegible] 1977.

vez necesite ser más delgada en clima caluroso, para asegurarse de que cubra la capa anterior, mientras ésta todavía se encuentra en condiciones de responder a la vibración. El intervalo entre los colados monolíticos de muros y losas de cubierta (con el fin de permitir que el muro de concreto desarrolle su contracción por asentamiento) se vuelve muy corto en clima caluroso, en especial cuando el concreto está tibio.

4.3.3 Al colar vigas y losas en clima caluroso es necesario mantener la operación reducida a una área pequeña y proceder con un frente que tenga una mínima cantidad de superficie expuesta a la cual deba añadirse el concreto. Por lo general, debe utilizarse una boquilla pulverizadora a fin de refrescar el aire, de enfriar las cimbras y el acero de refuerzo inmediatos, y de aminorar la evaporación rápida de la superficie del concreto, antes y después de cada operación de acabado. Debe evitarse un rociado excesivo (aquel que puede lavar la superficie del concreto u ocasionar encharcamiento en la superficie durante el pulido).

4.3.3.1 Si no se utiliza el rociado entre las operaciones de acabado en clima caluroso, en particular si hace mucho viento y es escasa la humedad, se puede provocar que la evaporación del agua de la superficie sea más rápida que la que surge naturalmente hacia ella. Esto creará una tensión creciente en la superficie que a menudo produce el agrietamiento irregular por contracción plástica. Se recomienda el uso cuidadoso del rocío del tipo de neblina antes mencionado, extender y remover el recurbimiento de polietileno entre las operaciones de acabado, o la aplicación de películas monomoleculares[28] después de descimbrar. En ocasiones, en algunos colados relativamente masivos, la revibración previa al pulido evitará la aparición de grietas por contracción plástica. Cuando dicho agrietamiento se presenta antes del fraguado final, es posible cerrar las grietas golpeando la superficie a cada lado de la grieta con una llana. No se obtiene ningún resultado duradero con sólo poner lechada sobre ellas y someterlas a un proceso de pulido.

4.3.4 En resumen, a fin de asegurar buenos resultados en la colocación del concreto en clima caluroso, la temperatura inicial de dicha colocación debe estar limitada a un punto, de preferencia entre los 24 y los 38°C, tal como se indica en las Secciones 2.2.2 y 4.2.4. Debe hacerse todo el esfuerzo posible para mantener uniforme la temperatura del concreto. Deben tomarse todas las medidas necesarias para colocar el concreto inmediatamente después de su llegada a la obra y de vibrarse al terminar su colocación. Las losas a nivel del terreno deben protegerse de un secado excesivo durante las operaciones de acabado y cada una de éstas debe realizarse, sin demora, en el momento en que el concreto esté listo para ello. En condiciones extremas de alta temperatura ambiental, exposición directa a los rayos del sol, baja humedad relativa y viento —tal [illegible] agr[illegible] po[illegible] lento ritmo de coloc[illegible], debido a la [illegible]-

plejo de la estructura por su tamaño o forma—, aun el cuidadoso y completo apego a las prácticas mencionadas puede no producir el grado de calidad deseado para el trabajo. En esas circunstancias, se ha encontrado que vale la pena restringir la colocación del concreto a las últimas horas de la tarde o al anochecer.

4.4 Curado y protección

4.4.1 En la Sección 4.2 se ha cubierto casi por completo el renglón relativo al curado y la protección. Debe hacerse hincapié en el hecho de que, en clima caluroso, existe una gran necesidad de tener un curado continuo, de preferencia por medio de agua. La necesidad se acrecienta durante las primeras horas, casi se puede decir que durante todo el primer dia, a partir de la colocación del concreto. Todas las superficies deben protegerse del secado, aun del intermitente, ya que todo esto contribuye al desarrollo del agrietamiento.

4.4.2 Cuando se trate de estructuras contenedoras de agua debe considerarse que las cimbras de maderas absorbentes que permanecen en su sitio, no son los medios adecuados para curar en clima caluroso y seco. Las cimbras deben cubrirse y mantenerse húmedas. Deben aflojarse tan pronto como esto pueda hacerse sin dañar el concreto y es necesario tomar medidas para que el agua de curado corra por dentro de ellas. Durante el descimbrado, deben tomarse precauciones para proporcionar cubiertas húmedas a las superficies recién expuestas con el fin de evitar su exposición directa a los rayos del sol y al viento. Los agujeros para sujetadores de cono se pueden rellenar y cualquier reparación que pudiera ser necesaria se puede efectuar descubriendo pequeñas porciones a cada vez, conforme vaya siendo necesario para continuar este trabajo. Dichas reparaciones deben quedar terminadas en el curso de los primeros dias después del descimbrado, de manera que las reparaciones y el relleno de los agujeros de los conos se puedan curar al mismo tiempo que el concreto que los rodea. Al finalizar el periodo de curado prescrito (el minimo es de 7 dias; 10 dias es preferible), la cubierta debe permanecer en su lugar sin volverse a mojar durante varios dias (se sugieren 4 dias), de manera que la superficie del concreto se seque lentamente y sea menos propensa al agrietamiento por contracción. Los efectos del secado se minimizan aún más si se aislan estructuras tales como túneles y tuberias, de las corrientes de aire y la libre circulación del aire seco.

4.4.3 En resumen, proporcionar la temperatura y las condiciones de humedad apropiadas para el curado del concreto es mucho más crítico e importante en climas calurosos, que a temperaturas normales. Por lo tanto, es de primordial importancia que se inicie el curado inmediatamente, que se cuente con amplia cobertura y que se continúe sin interrupción.

CAPITULO 5

pruebas e inspección

5.1 Pruebas

5.1.1 Las pruebas del muestreo de concreto fresco deben llevarse a cabo y los cilindros prepararse sin retrasos, de manera que resulten tan representativos como sea posible del concreto utilizado en la estructura. La alta temperatura, la baja humedad relativa y los vientos secos son dañinos para todos los concretos y en particular para el pequeño volumen de concreto utilizado para las pruebas y para moldear los cilindros. El dejar la muestra expuesta a los rayos del sol, al viento o al aire seco, puede afectar seriamente los resultados de las pruebas.

5.1.2 En ocasiones resulta deseable efectuar pruebas tales como la del revenimiento y la del contenido de aire con mayor frecuencia que las realizadas en condiciones normales; también pueden indicarse pruebas adicionales. Algunos ejemplos de tales pruebas adicionales son: temperaturas de los materiales y del concreto, tiempo de fraguado inicial y final, pérdida de revenimiento, y temperatura y humedad relativa en las cimbras.

5.1.3 Debe prestarse especial atención a la protección y al curado de cilindros moldeados para las pruebas de resistencia. Debido a lo reducido de su tamaño, en relación con la mayoria de las partes de la estructura, resulta fácil que los cilindros de prueba alcancen temperaturas más altas, sufran más cambios de temperatura, y se sequen completamente y con mayor rapidez que el concreto colado en la obra, con los correspondientes efectos dañinos aumentados. Por estas razones, en climas calurosos se necesita tener especial cuidado de mantener la temperatura uniforme y las condiciones de humedad apropiadas en los cilindros, que se requieren para cumplir con los métodos de prueba de la ASTM, para la prueba de resistencia. Las temperaturas adecuadas se pueden mantener evitando la exposición al sol y utilizando los efectos refrescantes de la evaporación del agua, ya sea por medio de tela de yute húmeda o de arena mojada que cubra los cilindros. En clima caluroso el simple acto de cubrir la parte superior del cilindro de prueba moldeado con una tapa, o una placa de metal, no resulta suficiente para evitar la pérdida de humedad y para conservar las temperaturas estándar. De hecho, es importante mantener agua libre en la superficie de los cilindros. Debe utilizarse arena mojada y tela de yute húmeda (que se mantienen húmedas) y rocío del tipo de la neblina, a fin de asegurarse de que los cilindros retengan el agua y evitar así alzas excesivas de temperatura.

5.1.4 Los cilindros de prueba utilizados como base para la aceptación del concreto al entregarse en una obra, deben estar protegidos del secado y de los incrementos de temperatura, y deben transferirse en condiciones de curado húmedo continuo estándar al laboratorio a la edad de un día. Durante su traslado también deben estar protegidos y es indispensable que se manejen con todo cuidado.

5.1.5 Se pueden hacer cilindros adicionales para curarse en la obra con objeto de que auxilien en la determinación de cuándo descimbrar, cuándo retirar los puntales y cuándo poner en servicio la estructura. A menos que los cilindros utilizados para estos propósitos sean curados en la misma obra y, tanto como sea posible, en condiciones semejantes a las de la estrucutra, dichos cilindros pueden ser engañosos.

5.2 Inspección

5.2.1 La Norma ACI 311 "Recommended Practice for Concrete Inspection", cubre los numerosos detalles a los cuales hay que apegarse en relación con la edificación de una buena obra de concreto. Los efectos particulares del clima caluroso en el comportamiento del concreto y las precauciones que se deben tomar con objeto de minimizar sus efectos adversos, se han discutido a fondo en este trabajo. La supervisión apropiada del concreto colado en clima caluroso debe enfocarse a asegurar el cumplimento de estos procedimientos y de estas precauciones adicionales. La inspección competente se anticipará a la necesidad de procedimientos tales como el rociado de las cimbras y de la sub-base; la necesidad de hielo como parte del agua de mezclado; el suministro de sombras, protectores contra el viento, o rociado del tipo de neblina, y otros

del mismo tipo; y minimizar los retrasos durante la colocación y el curado del concreto.

5.2.2 Los supervisores deben verificar la temperatura del aire, las condiciones ambientales generales (despejado, nublado), la velocidad del viento y la humedad relativa, a intervalos frecuentes. La pérdida de revenimiento y el agua añadida a la hora de llegada, al igual que el tiempo de mezclado correspondiente, siempre deben ser registrados. Los registros deben incluir la frecuente verificación de las temperaturas del concreto, observaciones acerca del comportamiento y la apariencia del mismo, a la hora de la entrega y después de haber sido colado en las cimbras, al igual que la protección, el tipo y el tiempo de curado según las especificaciones. Todos estos datos deben identificarse con el trabajo que se lleva a cabo, de modo que las condiciones que rodean la construcción de cualquier parte de la estructura se puedan determinar en fecha posterior, en caso de que esto fuera necesario. En los registros permanentes del proyecto debe incluirse una copia de todas estas observaciones.

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SEGUNDA PARTE

colocación del concreto en climas fríos

COLOCACION DEL CONCRETO EN CLIMAS FRIOS

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COLOCACION DEL CONCRETO EN CLIMAS FRIOS *
(ACI 306)

* Informe presentado por el American Concrete Institute en mayo de 1978 para sustituir a la norma ACI 306-66.

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sinopsis

Se comentan los requisitos generales para la producción de un concreto satisfactorio, al igual que los métodos para cumplir con dichos requisitos. Para muchos concretos estructurales se requiere un considerable exceso de protección, a fin de asegurarse de que estén libres del daño ocasionado por el congelamiento a temprana edad y así, garantizar el seguro desarrollo de la resistencia. Se discuten los acelerantes, el mantenimiento de registros de temperatura, el calentamiento de los materiales, la preparación de las sub-bases, las cubiertas aislantes de protección, los recintos con calefacción, el curado, el concepto de madurez y el descimbrado. Se menciona el material suplementario, en base a fuente autorizadas, acerca del efecto de las temperaturas de curado en la resistencia del concreto. Se incluye una lista de las referencias seleccionadas.

Palabras clave: aditivos; agentes acelerantes; agentes inclusores de aire; agregados; agua; aislamiento; calentamiento; cimbras (construcción); cloruro de calcio; **construcción con concreto; construcción en clima frío; concretos; concreto reforzado**; curado; descimbrado; durabilidad; durabilidad y el congelamiento y deshielo; edad; manejo de los materiales; resistencia a la compresión; sub-bases; temperatura.

CAPITULO 1

introducción

1.1 El objetivo primordial de esta guia es el de describir los procedimientos de construcción que garantizarán que el concreto colocado en clima frio dé como resultado una estructura lo suficientemente resistente y durable para satisfacer los requisitos de servicio. El concreto colocado bajo condiciones de clima frio desarrollará estas cualidades únicamente si ha sido hecho, colocado y protegido de manera adecuada. El grado necesario de protección aumenta, a medida que disminuye la temperatura ambiente. El clima frio se define como un periodo de tiempo en el cual durante más de 3 dias consecutivos la temperatura diaria cae por debajo de 5°C.

1.2 Los planes para proteger al concreto fresco del congelamiento y para mantener las temperaturas por encima del minimo diseñado, durante el tiempo requerido después del colado, deben prepararse de antemano cuando se esperan temperaturas de congelamiento. Todo el equipo y los materiales necesarios deben estar en la obra antes de que caiga la primera helada y no después de que el concreto ha sido coolcado, y su temperatura se acerque al punto de congelación.

1.3 En clima moderadamente frio, como el del otoño, cuando se ha pronosticado helada fuerte o temperaturas de congelamiento en la obra, todas las superficies de concreto descubiertas deben estar protegidas de la congelación, por lo menos durante las primeras 24 horas a partir de su colocación.* Cuando ha pasado el clima frio, debe proporcionarse una

* Todas las gráficas que indican las fechas promedio de temperaturas de congelamiento en los Estados Unidos se pueden obtener del National Climatic Center, Federal Building, Asheville, N. C. 28801.

protección similar contra el congelamiento. El concreto así protegido estará a salvo del daño por congelamiento a temprana edad y, si tiene aire incluido y después recibe un curado apropiado, seguido de un secado, no sufrirá daño en su durabilidad final. La protección contra el congelamiento durante las primeras 24 horas no garantiza el desarrollo de la resistencia requerida a la edad especificada, en particular cuando se espera clima considerablemente más frío. Por lo tanto, el curado y la protección deben continuar durante el tiempo necesario —y a la temperatura lo suficientemente elevada respecto al punto de congelación— para producir la resistencia requerida. Véanse los Capítulos 6 y 7.

1.4 En climas más fríos, cuando las temperaturas medias diarias por lo general se mantienen por debajo de los 5°C, el concreto debe colocarse a una temperatura no menor que la que se lee en la línea 1 de la Tabla 1.4.1, según la clase de concreto indicado. Es más, debe mantenerse a una temperatura no menor que esa durante el lapso indicado en la Tabla 1.4.2. Este periodo depende del tipo y de la cantidad de cemento, de si se ha utilizado un acelerante y de la condición de servicio. El concreto puede estar protegido contra el congelamiento, pero puede no haber desarrollado la capacidad adecuada para recibir carga. La Tabla 1.4.1 es particularmente aplicable a concreto de peso normal. El concreto ligero estructural es más resistente a la pérdida de calor que el concreto de peso normal, y ésta es significativamente menor bajo condiciones de resequedad. Por consiguiente, las temperaturas mínimas de la Tabla 1.4.1, correspondientes a la temperatura en el momento de la colocación y a aquélla a la cual se debe mantener, pueden bajarse, en el caso del concreto ligero, una vez que se haya efectuado la investigación de laboratorio requerida. La experiencia indica que el concreto ligero recién mezclado muestra una mayor retención de calor que el concreto de peso normal recién mezclado. Los concretos aislantes de peso más ligero demuestran una capacidad de retención de calor aún más alta. Cuando están expuestos a temperaturas de congelamiento, dichos concretos pueden ser especialmente susceptibles a sufrir de congelamiento en la superficie.

1.5 Los periodos de protección comparativamente cortos que se muestran en la Tabla 1.4.2, se aplican a concreto con aire incluido, con un contenido de aire como el recomendado por el Comité ACI 211 * y constituyen los requisitos mínimos de protección para la buena durabilidad de dicho concreto.** Estos cortos periodos sólo son permisibles

* Comité ACI 211. "Práctica recomendable para dosificar concreto normal y concreto pesado (ACI 211.1.74)", Instituto Mexicano del Cemento y del Concreto, A. C.,

** Puesto que el concreto sin aire incluido no se debe utilizar donde se presenta congelamiento y deshielo, este concreto no está cubierto por las recomendaciones; no obstante, la durabilidad potencial del concreto sin aire incluido resulta mejor asegurada

TABLA 1.4.1.— TEMPERATURAS RECOMENDADAS PARA EL CONCRETO

Línea	Temperatura del aire	Tamaño de la sección, dimensión mínima, m.			
		0.30	0.30 – 0.90	0.90 – 1.80	1.80
Temperatura mínima del concreto en el momento de la colocación y que se mantiene					
1	–	13°C	10°C	7°C	5°C
Temperatura mínima del concreto en el momento del mezclado según el clima indicado *					
2	Arriba de −1°C	16°C	13°C	10°C	7°C
3	− 18 a −1°C	18°C	16°C	13°C	10°C
4	−18°C	21°C	18°C	16°C	13°C
Máxima caída gradual de temperatura permisible durante las primeras 24 hrs después de terminada la protección					
5	–	28°C	22°C	17°C	11°C

*Para clima más frío se proporciona un margen mayor de temperatura entre el concreto en el momento del mezclado y la temperatura mínima requerida para la colocación de concreto fresco.

siempre y cuando ¹ haya suficiente curado posterior (véase el Capítulo 5) o tiempo de curado a bajas temperaturas, a fin de desarrollar la resistencia de diseño requerida en el momento en el cual se necesita, y siempre y cuando ² el concreto no esté sujeto a congelamiento en condiciones de saturación. Si se espera que algo del concreto quede expuesto al congelamiento, en condiciones de saturación, durante la construcción, debe contar con una adecuada inclusión de aire, aunque una vez en servicio, no sea expuesto a dicho congelamiento.

1.6 La verdadera temperatura de la superficie del concreto determina la efectividad de la protección, sin tomar en consideración las temperaturas del aire o si el objetivo es la durabilidad o la resistencia. A fin de evaluar y verificar la protección proporcionada, las temperaturas de las esquinas y de las orillas del concreto deben vigilarse (véase el Capítulo 9), dado que éstas son las partes más vulnerables al congelamiento y las más difíciles de mantener a la temperatura requerida.

por medio de, por lo menos, dos veces la longitud de protección indicada en la Tabla 1.4.2. En cuanto al concreto con aire incluido, en el cual se encuentran involucrados los criterios de resistencia, la protección debe ser la suficiente a fin de que el concreto alcan[illegible] resist[illegible] requ[illegible] para [illegible]rticular tipo de servicio.

TABLA 1.4.2.– PROTECCION RECOMENDADA PARA CONCRETO COLOCADO EN CLIMA FRIO*

Categoría del servicio	Protección recomendada en la temperatura que se indica en la línea 1, Tabla 1.4.1, días**			
	Contra daño por congelamiento***		Para resistencia segura	
	Cemento del Tipo I y II	Tipo III, acelerante o 20°/o extra de cemento	Cemento del Tipo I y II	Tipo III, acelerante o 20°/o extra de cemento
1. Sin carga, sin exposición (Véase la Sección 6.1.1)	2	1	2	1
2. Sin carga, expuesto (Véase la Sección 6.1.2)	3	2	3	2
3. Carga parcial, expuesto (Véase la Sección 6.1.3)	3	2	6	4
4. Carga completa, expuesto (Véase el Capítulo 7)	3	2	Véase la Tabla 7.7	

* Clima susceptible de tener una temperatura media diaria inferior a los 5°C. Véanse las Secciones 1.3 y 1.4.
** La protección debe interrumpirse únicamente de acuerdo con las instrucciones de la Sección 1.10.4.
*** El concreto masivo requerirá una protección más extensa como defensa contra el agrietamiento térmico por contracción y, cuando el contenido de cemento es bajo, requerirá una protección más extensa hasta que el concreto alcance una resistencia de 35 kg/cm2.

1.7 En caso de utilizarse, las cubiertas calentadas deben ser resistentes y a prueba de viento y de la intemperie. De lo contrario, a pesar de un alto consumo de combustible, no se pueden mantener las temperaturas adecuadas en las esquinas, las orillas y en secciones delgadas de concreto. No debe permitirse que los equipos de calefacción calienten o sequen el concreto en un punto determinado. El concreto fresco, expuesto al bióxido de carbono (CO_2) procedente de atmósferas contaminadas, o que es el resultado del uso de parrillas o de otros dispositivos que producen gases debido a la combustión directamente hacia una zona encerrada, puede producir la carbonatación del concreto, provocando superficies suaves de profundidad variable que depende de la concentración de CO_2, de la temperatura de curado del concreto y de la

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humedad relativa. El monóxido de carbono, que se presenta debido a la combustión parcial, al igual que las altas concentraciones de CO_2, representan un peligro potencial para los trabajadores. Más aún, deben ponerse en vigor medidas estrictas para la prevención de incendios. El fuego puede dañar el concreto a cualquier edad, pero a una edad muy temprana, puede sufrir un daño adicional debido al congelamiento, si éste se presenta antes de proporcionarle una nueva protección.

1.8 El concreto con revenimiento más bajo que el normal (< 10 cm) resulta particularmente deseable para losas planas en clima frio: de esta manera, se minimiza el agua del sangrado y el fraguado se presenta con mayor rapidez. Si el agua de sangrado permanece mucho tiempo sobre la superficie, puede llegar a impedir la obtención de un buen acabado y producir una superficie suave y polvosa.

1.9 El concreto siempre debe ser colocado a las temperaturas más cercanas a las temperaturas más bajas permisibles (véase la Tabla 1.4.1). Las temperaturas que excedan de la indicada en la Tabla 1.4.1, en más de 5°C deben ser evitadas. Debe aprovecharse la oportunidad que, a colocar concreto a baja temperatura, se presenta con el clima frio. El concreto, al cual no se le permite congelarse, que se coloca a bajas tempeturas por encima del punto de congelación (5 a 13°C) y que recibe un curado a largo plazo, desarrolla una resistencia última más alta, mayor durabilidad y está menos sujeto al agrietamiento térmico, que un concreto similar colado a temperaturas más altas. La alta temperatura del concreto en el momento de la colocación aminorará sus buenas propiedades, aunque puede apresurar los acabados en clima frio.

1.10 La colocación de concreto durante el invierno debe apegarse a lo siguiente:

1.10.1 Evitar daños al concreto debidos al congelamiento a edad temprana. El grado de saturación del concreto recién colocado, que carece de acceso a fuentes exteriores de agua, se reducirá a medida que se endurezca el concreto y se utilice agua durante el proceso de hidratación. En estas condiciones, el tiempo en el cual se reduce el grado de saturación, por debajo del nivel que podría ocasionar daños por congelamiento, corresponde aproximadamente al tiempo que toma el concreto para alcanzar una resistencia a la compresión de 35 kg/cm². A una temperatura de 10°C, la mayoría de los concretos bien proporcionados alcanzarán esta resistencia durante el segundo día.

1.10.2 Permitir que el concreto desarrolle cualquier resistencia que sea necesaria, según su tipo de servicio (Tabla 1.4.2) con objeto de garantizar un descimbrado seguro para una rápida reutilización de la cimbra, el retiro seguro de los puntales y el reapuntalamiento, al igual que poder cargar con seguridad la estructura cuando se desee, en el curso de la construcción y después de haberla terminado.

1.10.3 Mantener las condiciones de curado que propiciarán el desarrollo normal de la resistencia, libres de calor excesivo y de saturación crítica del concreto, al llegar a su fin el periodo de protección (véase el Capítulo 7).

1.10.4 Limitar los cambios bruscos de temperatura, en particular antes de que la resistencia se haya desarrollado lo suficiente para soportar los esfuerzos por temperatura. El enfriamiento repentino de las superficies de concreto, o de los miembros exteriores, en relación con la estructura interior, puede provocar agrietamiento en detrimento de la resistencia y la durabilidad. Al finalizar el periodo requerido, debe removerse la protección de manera tal, que el descenso de la temperatura de cualquier porción del concreto sea *gradual* y no exceda, en el término de 24 horas, de la cantidad indicada en la linea 5 de la Tabla 1.4.1, según el tamaño de la sección de concreto. Esta rapidez puede lograrse por medios tales como la disminución paulatina de las fuentes de calor, o permitiendo que el aislamiento permanezca hasta que el concreto haya alcanzado un equilibrio de temperatura con la del medio ambiente.

1.10.5 Proporcionar protección congruente con el tipo de servicio al que se destina la estructura. Las estructuras de concreto están destinadas a una vida de muchos años. Si la estructura muestra esquinas dañadas por el congelamiento, zonas deshidratadas, o grietas debidas al sobrecalentamiento, que no son sino el resultado de una falta de protección adecuada, un curado inapropiado y una supervisión descuidada, no suele ser satisfactoria la resistencia obtenida de cilindros a los 28 días. De manera similar, la resistencia inicial de una estructura, lograda mediante el uso indiscriminado de demasiado cloruro de calcio, no servirá si, años más tarde, el concreto sufre un severo agrietamiento debido a una expansión interna destructiva, como resultado de una reacción álcali-agregado aumentada, o la corrosión del acero de refuerzo. No se debe tratar de hacer economías en la construcción mediante el sacrificio de la durabilidad. Véase la publicación ACI 201 "Guide to Durable Concrete".

La experiencia ha demostrado que el costo de una adecuada protección para el concreto colado en clima frío, no tiene por qué considerarse excesivo, tomando en cuenta lo requerido y el beneficio que se obtiene. Sin embargo, sólo el propietario puede decidir si el costo extra de la operación de colocación de concreto en clima frío será una inversión provechosa. La alternativa puede ser la de esperar a que mejore la temperatura. Puesto que la negligencia en la protección temprana contra el congelamiento puede tener como resultado la inmediata destrucción o la permanente debilidad del concreto, es evidente que la protección adecuada contra las bajas temperaturas y el curado apropiado en todas las operaciones de colocación de concreto en clima frío son esenciales.

CAPITULO 2

temperatura del concreto en el momento de la colocación -calentamiento de los materiales

2.1 La protección descrita en el Capítulo 4 debe proporcionarse inmediatamente después de la colocación del concreto, con el fin de asegurarse de que no sufra congelamiento antes de que haya sido posible instalar la protección. La temperatura del concreto en el momento de la colocación no debe ser inferior a la que se indica en la linea 1 de la Tabla 1.4.1. Las altas temperaturas en el concreto no ofrecen proporcionalmente una mayor protección contra el congelamiento, ya que la pérdida de calor es más rápida en diferenciales mayores de temperatura. Además, a temperaturas más elevadas se requiere más agua de mezclado, por lo que aumenta la pérdida de revenimiento, en ocasiones se provoca un fraguado rápido y se incrementa la contracción térmica. La rápida pérdida de humedad en las superficies de concreto expuestas, puede provocar agrietamientos por contracción plástica. Por lo tanto, la temperatura del concreto fresco, en el momento de la colocación, debe mantenerse tan cerca de las temperaturas mínimas sugeridas como sea posible.

2.2 El concreto en el momento del mezclado debe mantenerse a temperaturas no más altas de 6°C sobre las mínimas recomendadas en la Tabla 1.4.1. Deben evitarse las temperaturas superiores a los 11°C sobre estos valores. En tanto que resulta difícil calentar los agregados de manera uniforme a una temperatura predeterminada, la temperatura del agua de mezclado puede ajustarse fácilmente mezclando agua caliente y agua fría, con el fin de obtener concreto dentro de los 6 C de la temperatura requerida.

2.3 Conforme desciende la temperatura del aire, son más los ingredientes que deben calentarse con objeto de producir concreto recién mezclado a la temperatura mínima deseada. Durante los periodos más fríos, la temperatura del concreto debe elevarse a fin de anular la pérdida de calor ocurrida en la etapa entre el mezclado y la colocación, tal como se recomienda en las líneas 2, 3 y 4 de la Tabla 1.4.1. Mientras más masiva sea la sección de concreto, más lentamente perderá calor. En la línea 1 de la Tabla 1.4.1 se recomiendan las temperaturas mínimas y más bajas para el concreto, en el momento de la colocación, a medida que la sección de concreto sea más masiva. En obras masivas, el concreto se beneficia, de manera muy particular, por las bajas temperaturas iniciales.

2.4 Cuando los agregados están libres de hielo y de grumos congelados, por lo general se puede obtener la temperatura deseada del concreto, calentando sólo el agua de mezclado, pero cuando de manera consistente, las temperaturas del aire se mantienen por debajo de 0°C, resulta necesario calentar los agregados. Si el agua de mezclado está a 60°C, sólo en contadas ocasiones es necesario calentar los agregados a temperturas de más de 15°C. Si el agregado grueso está seco y libre de escarcha, hielo y grumos congelados, se pueden obtener las temperaturas adecuadas del concreto fresco por medio del incremento en la temperatura de la arena únicamente, la cual, sólo en raras ocasiones tendrá que ser superior a los 40°C aproximados, si el agua de mezclado está a 60°C. Asimismo, deben tenerse en consideración las variaciones estacionales, dado que las temperaturas promedio de los agregados pueden ser sustancialmente más altas que las temperaturas del aire en el otoño, ocurriendo lo contrario en la primavera.

2.5 El agua de mezclado debe estar disponible a una temperatura regulada y consistente, y en cantidades suficientes, a fin de evitar fluctuaciones de consideración en la temperatura y, por consiguiente, en el revenimiento del concreto entre una dosificación y otra.

Por medio de informes se ha sabido que el contacto prematuro entre agua sumamente caliente y cantidades concentradas de cemento provoca un fraguado relámpago y la formación de grumos de cemento en los camiones mezcladores. Cuando se utiliza agua a una temperatura superior a 60°C, puede resultar necesario ajustar el orden en el cual se mezclan los ingredientes. Puede servir de ayuda, si se inicia con el agua caliente y el agregado grueso, antes que el cemento y se detiene o aminora la rapidez de entrada del agua, mientras se cargan el cemento y el agregado.

Si es necesario dosificar el cemento, separadamente de los agregados, el mezclado puede resultar más difícil. En este caso, deben colocarse aproximadamente tres cuartas partes de la adición de agua caliente en

el tambor, ya sea antes de los agregados, o al mismo tiempo que éstos. El cemento debe añadirse después de los agregados. La cuarta parte restante del agua debe cargarse por la boca de descarga del tambor a un ritmo moderado como ingrediente final.

Se puede utilizar agua a la temperatura del punto de ebullición, siempre que las temperaturas del concreto resultante estén dentro de los límites descritos en la Sección 2.2 y que no se presente un fraguado relámpago. Si se llegara a notar una pérdida de efectividad del aditivo inclusor de aire, debida al contacto inicial con el agua caliente, dicho aditivo debe agregarse a la carga una vez que la temperatura del agua haya disminuido por el contacto con los materiales sólidos más frescos.

2.6 El calentamiento de los agregados debe efectuarse de manera tal, que no haya grumos de hielo, de nieve o de agregado. Con frecuencia sobrevivirán al mezclado algunos grumos congelados de 75 mm y permanecerán en el concreto después de la colocación de éste. Debe evitarse el sobrecalentamiento de manera que las temperaturas de aquellos agregados, cuyas cantidades en la dosificación son reducidas, no excedan de 100°C y que el promedio de temperatura del volúmen de la carga no exceda de 65°C. Cualquiera de estas temperaturas es considerablemente superior a la necesaria para obtener temperaturas deseables en el concreto recién mezclado. Los materiales deben calentarse de manera uniforme, ya que una variación considerable en su temperatura hará variar notablemente el requerimiento de agua, la rapidez del endurecimiento y el revenimiento del concreto.

Después de un periodo prolongado de calentamiento con vapor de los agregados en los almacenes de depósito, se requiere de un cuidado especial para la dosificación de las primeras cargas de concreto. Muchos productores de concreto no utilizan las primeras toneladas de agregado extremadamente caliente. Este material, normalmente se descarga y se somete a un reciclaje, colocándolo encima del agregado que ha permanecido en los almacenes de depósito.

2.7 Con objeto de calentar el agregado se recomienda la circulación del vapor por tuberías, pero para obras pequeñas, los agregados se pueden descongelar calentándolos cuidadosamente sobre tubería de fierro fundido dentro de la cual se mantiene fuego encendido. Cuando los agregados se descongelan o se calientan, por medio del vapor que circula en tuberías, las superficies expuestas de éstos deben cubrirse con lonas, en la medida de lo posible, a fin de mantener una distribución de calor uniforme y de evitar la formación de superficies de agregado congeladas. Si el vapor está confinado en un sistema de calefacción por tuberías, resulta posible evitar las dificultades que se generan en la humedad variable de los agregados, pero, a la vez, aumenta la posibilidad de puntos localizados secos y calientes. Los chorros de vapor liberados en el agregado pueden ocasionar problemas de variación en la humedad. Eventualmente, el desgaste y la corrosión en la tubería por donde pasa el vapor hacia los agregados provocará fugas que pueden conducir al mismo problema de variación en la humedad, provocada por los chorros de vapor. Se recomienda llevar a cabo inspecciones en la tubería y realizar los reemplazos necesarios.

2.8 Cuando las circunstancias requieren del deshielo de una cantidad considerable de material que está a temperatura extremadamente baja los chorros de vapor pueden constituir el único medio práctico de proporcionar el calor necesario. En este caso, el deshielo debe efectuarse con tanta anticipación como sea posible respecto a la dosificación, de manera de poder alcanzar un equilibrio sustancial tanto en el contenido de humedad, como en la temperatura. De allí en adelante, el suministro de vapor puede reducirse a un mínimo que evitará un nuevo congelamiento, minimizando, de esta manera y hasta cierto punto, los problemas que pudieran surgir de un contenido variable de humedad. No obstante, en esas condiciones, el control del agua de mezclado tendrá que estar sujeto a un ajuste individual para cada dosificación. Con el fin de mantener al agregado libre de hielo, se ha utilizado calor seco en vez de vapor.

2.9 Si se conocen los pasos y las temperaturas de todos los componentes y el contenido de humedad de los agregados, la temperatura final de la mezcla de concreto puede calcularse por medio de la siguiente fórmula:

$$T = \frac{0.22(T_c W_c + T_a W_a + T_g W_g) + T_m W_m + T_a W_{wa} + T_g W_{wg}}{0.22(W_c + W_a + W_g) + W_m + W_{wa} + W_{wg}}$$

donde

T = temperatura final de la mezcla de concreto (°C).

T_c, T_a, T_g, T_m, son las temperaturas en °C del cemento, del agregado fino, del agregado grueso y del agua de mezclado, respectivamente.

W_c, W_a, W_g, W_m, W_{wa}, W_{wg}, son los pesos (kg) de cemento, agregado fino y agregado grueso, excluyendo el contenido de humedad, agua y agua libre del agregado fino y del agregado grueso, respectivamente.

Si la temperatura de uno, o de ambos agregados es inferior a 0°C, la humedad se encontrará en estado de congelación y la fórmula arriba citada tendrá que ser modificada, a fin de tomar en cuenta el calor que se requiere para elevar la temperatura del hielo a 0°C y, de este modo, convertirlo en agua. En dicha fórmula, las expresiones $T_a W_{wa}$ y/o $T_g W_{wg}$ deben cambiarse por:

W_{wa} $(0.50\ T_a - 80)$ y/o W_{wg} $(0.50\ T_g - 80)$, respectivamente, según el caso (0.50 es el calor específico del hielo, y el número 80 se relaciona con el calor de fusión necesario para derretir el hielo).

2.10 El Instituto Sueco para Investigaciones del Cemento y el Concreto (Referencia 23) realizó pruebas con objeto de determinar la magnitud del descenso de temperatura del concreto que debía esperarse durante su entrega en clima frío. Sus estudios incluyeron mezcladoras de tambor giratorio, camiones de volteo abiertos y camiones de volteo cubiertos. El descenso de la temperatura para un lapso de una hora en el tiempo de entrega puede calcularse mediante las ecuaciones aproximadas que se dan a continuación:

1. Mezcladora de olla giratoria:

$$T = 0.25(t_r - t_a)$$

2. Camiones de volteo cubiertos:

$$T = 0.10(t_r - t_a)$$

3. Camiones de volteo abiertos:

$$T = 0.20(t_r - t_a)$$

donde

T = descenso de temperatura que se espera durante la entrega, °C (este valor debe sumarse a t_r, a fin de determinar la temperatura requerida para el concreto en la planta).

t_r = tempeatura requerida para el concreto en la obra, °C.

t_a = temperatura ambiente del aire, °C.

Los siguientes ejemplos ilustran la aplicación de estas reglas aproximadas:

1. El concreto debe mantenerse en continua agitación en una mezcladora de olla giratoria durante un periodo de entrega de una hora. La temperatura del aire es de −1°C y la del concreto, en el momento de la entrega debe estar, por lo menos, a 16°C. De la fórmula 1:

$$T = 0.25[16 - (-1)] = 4.25°C$$

Por lo tanto, debe darse tolerancia para un descenso de temperatura a 4.25°C, y el concreto en la planta debe tener una temperatura de, por lo menos, 20 + 4.25°C, en números redondos, 25°C.

2. Para las mismas condiciones que las del ejemplo 1, el concreto se puede entregar dentro del lapso de una hora y no hacer girar la olla excepto para el mezclado, y brevemente al momento de descarga. La aplicación de la constante de la fórmula 3 arriba citada, como la que mejor representa la situación da:

$$T = 0.20[16 - (-1)] = 3.4°C$$

Se tendrá que permitir una tolerancia para una temperatura de concreto de sólo, aproximadamente, 20 + 3.4 ó 24°C en la planta.

La ventaja de los camiones de volteo sobre las ollas giratorias sugiere que la pérdida de temperatura de éstos se puede minimizar evitando hacer girar la olla, excepto cuando sea absolutamente necesario, durante la entrega. La eliminación de la agitación, en la medida de lo posible, reduce la tolerancia requerida de pérdida de calor, por lo menos, hasta un nivel comparable al de los camiones de volteo.

CAPITULO 3

preparación previa a la colocación del concreto

3.1 La preparación para la colocación del concreto, aparte de lo que se describió en la Sección 1.2, consiste principalmente en asegurarse de que todas las superficies que vayan a estar en contacto con el concreto recién colocado estén a una temperatura que no pueda ocasionar un congelamiento prematuro, o prolongar severamente el endurecimiento. Por lo general, la temperatura de estas superficies de contacto, incluyendo los materiales de la sub-base, no necesita ser superior a unos cuantos grados sobre el punto de congelación, digamos 2°C, y de preferencia, no debe ser superior a la temperatura del concreto que se va a colocar, tal como se describe en la Tabla 1.4.1, línea 1.

Todo el hielo la nieve y la escarcha deben quitarse, de manera que no ocupen el espacio destinado al concreto sólido. Se pueden utilizar chorros de aire caliente para remover la escarcha, la nieve y el hielo de las cimbras, el acero de refuerzo y otros aditamentos ahogados. A menos que el área de trabajo esté cubierta, puede ser necesario llevar a cabo este trabajo justo en el momento anterior a la colocación del concreto con el fin de evitar que se vuelvan a congelar.

Nunca debe colocarse concreto sobre material congelado de la superficie de la sub-base. En ocasiones, la sub-base se puede descongelar lo suficiente como para colocar el concreto cubriéndola con material aislante durante unos días antes de la colocación del concreto, pero en la mayoría de los casos resulta necesaria la aplicación de calor externo.

Los experimentos en la obra mostrarán cuáles son las combinaciones de material aislante y de tiempo que permitirán que el calor superficial descongele el material de la sub-base. De ser necesario, el material descongelado debe volverse a compactar.

CAPITULO 4

protección

4.1 Antes de la colocación del concreto deben hacerse arreglos para cubrir, aislar, abrigar o calentar el concreto recién colocado, los cuales deben ser adecuados para alcanzar la temperatura y las condiciones de humedad que aquí se recomiendan en todas las zonas del concreto. En clima frío, la temperatura del concreto recién colocado debe mantenerse tan cercana como sea posible a los valores descritos en la línea 1 de la Tabla 1.4.1 para los periodos de tiempo indicados en la Tabla 1.4.2.

4.2 Puesto que la mayor parte del calor de hidratación del cemento en proceso de endurecimiento, se desarrolla durante los primeros 3 días, puede no requerirse calor de fuentes externas con objeto de mantener al concreto a las temperaturas correctas, si el calor que se genera en él se conserva de la manera adecuada. Este calor se puede retener por medio del uso de mantas aislantes sobre las superficies sin cimbras y de cimbras aislantes.[2] Algunos de los materiales que más se acostumbran son:

4.2.1 *Hojas de espuma de poliestireno:* Estas se pueden recortar en la forma deseada y fijar entre los pernos de las cimbras, o colocarlas en el lugar preciso por medio de pegamento.

4.2.2 *Espuma de uretano:* Esta espuma se puede rociar en la parte exterior de las cimbras con objeto de formar una capa aislante continua. La espuma de uretano debe, a su vez, ser rociada con un buen esmalte para exteriores, a fin de eliminar virtualmente la absorción de agua y protegerla del efecto dañino de los rayos ultravioleta. El empleo de espuma de uretano debe emprenderse con precaución, dado que, cuando entra en contacto con el fuego, genera gases sumamente tóxicos.

4.2.3 *Mantas de espuma de vinilo:* Este material consiste en mantas flexibles de espuma de vinilo con un lado recubierto de vinilo extruido. Para obtener calor adicional pueden también tener alambres eléctricos incorporados en la espuma. Las mantas no eléctricas se pueden obtener en rollos de ancho estándar. Las eléctricas deben mandarse hacer por pedido.

4.2.4 *Fibras de celulosa o lana mineral:* Por regla general, la lana mineral, o las fibras de celulosa, están recubiertas por gruesos forros de polietileno para formar grandes tapetes o rollos. A los forros de plástico en algunas ocasiones se les da un acabado superficial rugoso, a fin de reducir el riesgo de deslizamiento. Se pueden utilizar extendidos para cubrir las losas o plegados para envolver a otros elementos.

4.2.5 *Paja:* La paja sigue siendo popular, a pesar de no ser tan efectiva como las mantas o los tapetes. Algunas de sus desventajas son: su voluminosidad, su inflamabilidad y la necesidad de protegerla de la humedad. Las lonas alquitranadas, las películas de polietileno plástico, o el papel impermeable se deben utilizar como recubrimiento protector con objeto de disminuir las infiltraciones de aire, y de mantener la paja seca y en su lugar.

Para obtener el mayor provecho del uso de las mantas comerciales o del aislante de guata, y la más grande eficiencia al volverlos a utilizar, éstos deben estar bien protegidos por medio de un material de recubrimiento fuerte e impermeable a prueba de viento, lluvia, nieve o cualquiera otra forma de humedad, que pudiera perjudicar su valor aislante. El aislante debe mantenerse en estrecho contacto con las superficies del concreto o de las cimbras.

4.3 Los registros de temperatura del concreto revelan la efectividad de las distintas cantidades o clases de aislante o de otros métodos de protección, para diversos tipos de obras de concreto bajo diferentes condiciones climáticas. La selección y modificaciones adecuadas pueden realizarse de acuerdo con ellos. Ya han sido publicados algunos métodos adicionales para calcular las temperaturas que se pueden mantener, por medio de diversos arreglos para los aislantes, en determinadas condiciones climáticas.[2, 11] En base a los requerimientos de esta práctica recomendable, las Tablas 4.3.1, 4.3.2, 4.3.3 y 4.3.4, al igual que las figuras 4.3.1, 4.3.2, 4.3.3 y 4.3.4, indican la resistencia térmica, *R*, del aislante que se requiere para las caras de los muros o de las losas de concreto de distinto espesor, con diferente temperatura del aire y para diversos contenidos de cemento. Se ha supuesto que la temperatura del concreto en el momento de la colocación es de 10°C.

TABLA 4.3.1.– AISLANTE TERMICO PROPORCIONADO PARA MUROS DE CONCRETO Y LOSAS SOBRE EL TERRENO

Concreto colocado a 10°C. Protección mínima de 7 días

Espesor del muro o de la losa, m	Temperatura ambiente mínima del aire, grados C, permisible cuando se utiliza aislante con estos valores de resistencia térmica R, °C/W/m²			
	R = 0.35	R = 0.70	R = 1.06	R = 1.41
	Contenido de cemento = 178 kg/m³			
0.15	9	8	6	4
0.30	7	4	0	−4
0.46	5	−1	−6	−12
0.61	3	−4	−12	−19
0.91	0	−11	−22	−33
1.20	−3	−16	−27	−38
1.50	−3	−16	−27	−38
	Contenido de cemento = 237 kg/m³			
0.15	8	7	4	2
0.30	6	2	−3	−8
0.46	4	−4	−12	−19
0.61	1	−8	−19	−29
0.91	−4	−18	−31	−47
1.20	−8	−23	−39	•
1.50	−8	−23	−39	•
	Contenido de cemento = 296 kg/m³			
0.15	8	6	3	1
0.30	6	−1	−7	−13
0.46	2	−7	−17	−26
0.61	−1	−14	−27	−39
0.91	−8	−26	−43	−62
1.20	−12	−32	−51	•
1.50	−12	−32	•	•
	Contenido de cemento = 356 kg/m³			
0.15	8	5	2	−2
0.30	4	−2	−10	−18
0.46	1	−11	−22	−34
0.61	−3	−18	−33	−48
0.91	−11	−31	−54	•
1.20	−16	−40	•	•
1.50	−16	−40	•	•

• ≪ −51°C

TABLA 4.3.2.– AISLANTE TERMICO PROPORCIONADO PARA MUROS DE CONCRETO Y LOSAS SOBRE EL TERRENO

Concreto colocado a 10°C. Protección mínima de 3 días

Espesor del muro o de la losa, m	Temperatura ambiente mínima del aire, grados C, permisible cuando se utiliza aislante con estos valores de resistencia térmica R, °C/W/m²			
	R = 0.35	R = 0.70	R = 1.06	R = 1.41
Contenido de cemento = 178 kg/m³				
0.15	8	5	2	0
0.30	5	−1	−6	−12
0.46	2	−6	−13	−21
0.61	−1	−10	−19	−29
0.91	−3	−13	−26	−38
1.20	−3	−16	−27	−38
1.50	−3	−16	−27	−38
Contenido de cemento = 237 kg/m³				
0.15	7	3	0	3
0.30	3	−4	−11	−18
0.46	−1	−11	−21	−31
0.61	−4	−17	−29	−42
0.91	−7	−23	−38	−53
1.20	−8	−23	−39	•
1.50	−8	−23	−39	•
Contenido de cemento = 296 kg/m³				
0.15	6	2	−2	−7
0.30	1	−8	−16	−24
0.46	−4	−16	−29	−42
0.61	−8	−23	−39	−56
0.91	−11	−31	−51	•
1.20	−12	−32	•	•
1.50	−12	−32	•	•
Contenido de cemento = 356 kg/m³				
0.15	5	0	−5	−10
0.30	−1	−11	−22	−32
0.46	−6	−22	−37	−53
0.61	−12	−31	−51	•
0.91	−16	−38	•	•
1.20	−16	−40	•	•
1.50	−16	−40	•	•

• << −51°C

TABLA 4.3.3.– AISLANTE TERMICO PROPORCIONADO PARA LOSAS DE CONCRETO Y RECUBRIMIENTOS PARA CANALES SOBRE EL TERRENO

Concreto colocado a 10°C sobre terreno a 2°C. Protección mínima de 7 días

Peralte de la losa, m	Temperatura ambiente mínima del aire, grados C, permisible cuando se utiliza aislante con estos valores de resistencia térmica R, °C/W/m²			
	R = 0.35	R = 0.70	R = 1.06	R = 1.41
Contenido de cemento = 178 kg/m³				
0.10	•	•	•	•
0.20	•	•	•	•
0.31	•	•	•	•
0.46	8	6	2	−1
0.61	4	−1	−6	−12
0.76	2	−6	−14	−22
0.91	−1	−11	−21	−31
Contenido de cemento = 237 kg/m³				
0.10	•	•	•	•
0.20	•	•	•	•
0.31	•	•	•	10
0.46	5	0	−6	−11
0.61	2	−7	−17	−26
0.76	−2	−13	−26	−38
0.91	−5	−20	−34	−48
Contenido de cemento = 296 kg/m³				
0.10	•	•	•	•
0.20	•	•	•	•
0.31	9	7	4	2
0.46	2	−6	−13	−21
0.61	−2	−14	−27	−39
0.76	−6	−22	−38	−53
0.91	−9	−28	−46	•
Contenido de cemento = 356 kg/m³				
0.10	•	•	•	•
0.20	•	•	•	•
0.31	7	3	0	−3
0.46	−1	−10	−21	−31
0.61	−6	−21	−36	−52
0.76	−10	−28	−55	•
0.91	−14	−34	•	•

• > 10°C.: calor adicional requerido
• << −51°C

TABLA 4.3.4.– AISLANTE TERMICO PROPORCIONADO PARA LOSAS DE CONCRETO Y RECUBRIMIENTOS PARA CANALES SOBRE EL TERRENO

Concreto colocado a 10°C sobre terreno a 2°C. Protección mínima de 3 días

Peralte de la losa, m	Temperatura ambiente mínima del aire, grados C, permisible cuando se utiliza aislante con estos valores de resistencia térmica R, °C/W/m²			
	R = 0.35	R = 0.70	R = 1.06	R = 1.41
Contenido de cemento = 178 kg/m³				
0.10	•	•	•	•
0.20	•	•	•	•
0.31	•	•	•	•
0.46	6	3	0	−3
0.61	3	−4	−12	−19
0.76	−1	−9	−18	−27
0.91	−1	−11	−21	−30
Contenido de cemento = 237 kg/m³				
0.10	•	•	•	•
0.20	•	•	•	•
0.31	8	7	6	4
0.46	2	−6	−13	−21
0.61	−2	−13	−23	−34
0.76	−6	−18	−29	−41
0.91	−6	−20	−34	−46
Contenido de cemento = 296 kg/m³				
0.10	•	•	•	•
0.20	•	•	•	•
0.31	6	2	−1	−4
0.46	−1	−11	−21	−30
0.61	−6	−21	−35	−46
0.76	−9	−23	−41	−59
0.91	−9	−28	−46	•
Contenido de cemento = 356 kg/m³				
0.10	•	•	•	•
0.20	•	•	•	•
0.31	3	−3	−10	−17
0.46	−4	−18	−31	−44
0.61	−10	−27	−43	−63
0.76	−12	−29	−52	•
0.91	−14	−34	•	•

•) 10°C : calor adicional requerido
• ≪ −51°C

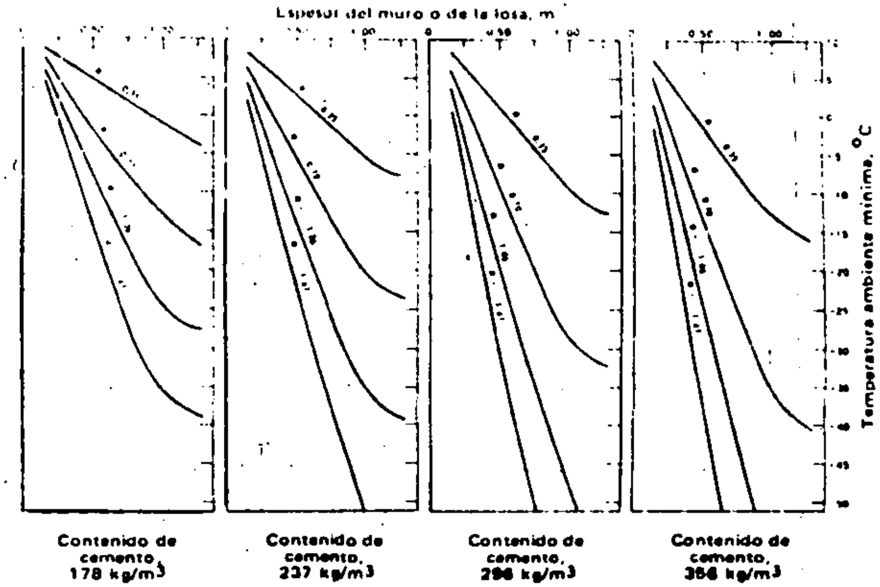

Figura 4.3.1.– Resistencia térmica R, en °C/W/m², del aislante requerida para muros de concreto y losas sobre el terreno. Concreto colocado a 10°C. Protección mínima de 7 días.

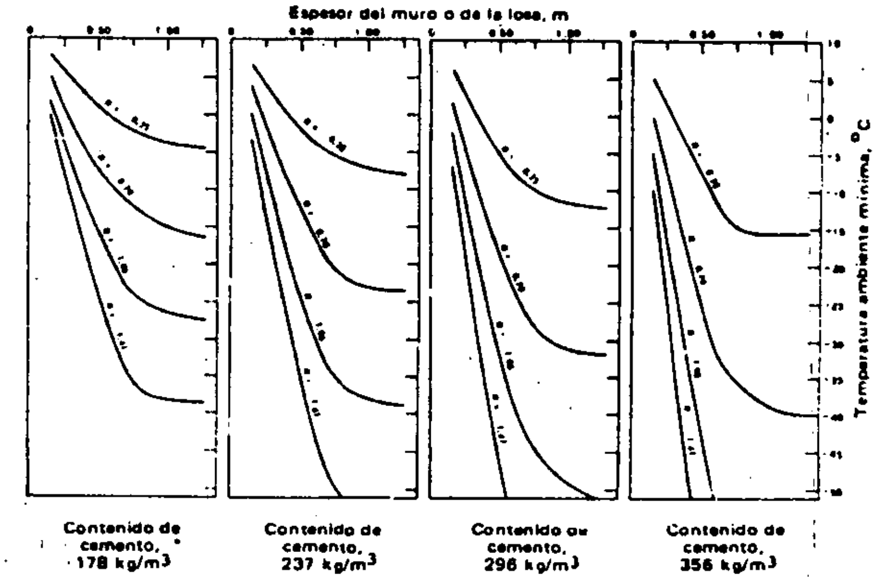

Figura 4.3.2.– Resistencia térmica R, en °C/W/m², del aislante requerida para muros de concreto y losas sobre el terreno. Concreto colocado a 10°C. Protección mínima de 3 días.

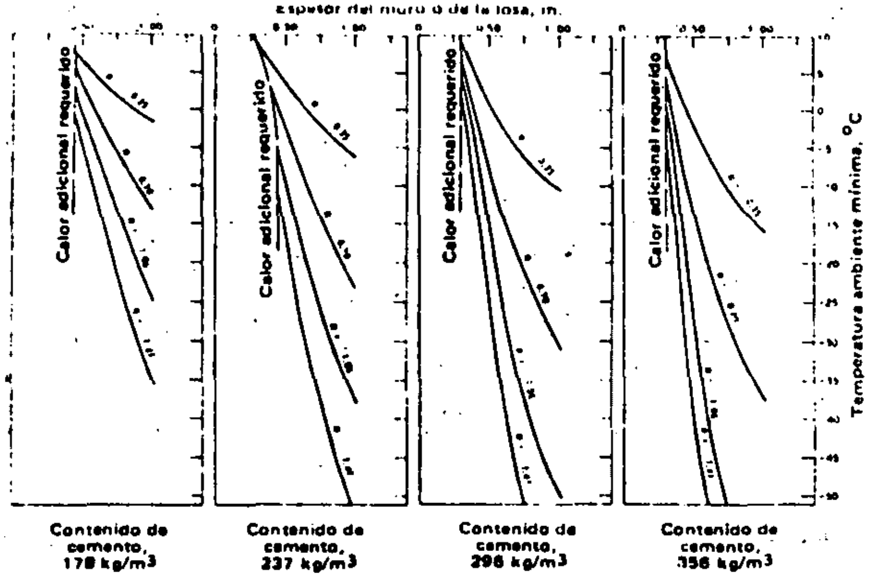

Figura 4.3.3.– Resistencia térmica R, en °C/W/m² del aislante requerido para losas de concreto y recubrimiento para canales sobre el terreno. Concreto colocado a 10°C sobre el terreno a 2°C. Protección mínima de 7 días.

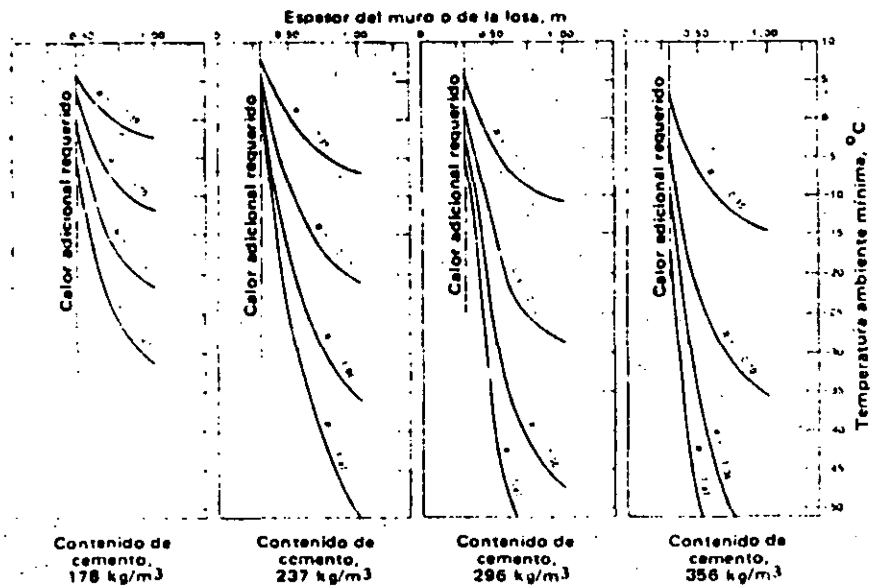

Figura 4.3.4.– Resistencia térmica R, en °C/W/m² del aislante requerido para losas de concreto y recubrimientos para canales colocados sobre el terreno. Concreto colocado a 10°C sobre terreno a 2°C. Protección mínima de 3 días.

TABLA 4.3.5.– VALORES DE AISLAMIENTO DE DIVERSOS MATERIALES

Material aislante	Resistencia térmica, R, para materiales con un espesor de 10 mm*
	°C/(W/m²)
Tablas y losas	
Poliuretano expandido (R-11 exp.)	0.433
Poliestireno expandido extruido (R-12 exp.)	0.347
Poliestireno expandido extruido, liso	0.277
Fibra de vidrio, unida orgánicamente	0.277
Poliestireno expandido, esferas moldeadas	0.248
Fibra mineral con adhesivo resinoso	0.239
Tabla de fibra mineral, con fieltro húmedo	0.204
Recubrimiento, densidad regular	0.182
Vidrio celular	0.173
Cartón laminado	0.139
Tabla de conglomerados (baja densidad)	0.128
Triplay	0.087
Mantas	
Fibra mineral, formas fibrosas procesadas a partir de roca, escoria o vidrio	0.224
Relleno suelto	
Fibra de madera, maderas suaves	0.231
Fibra mineral (roca, escoria o vidrio)	0.216
Perlita (expandida)	0.187
Vermiculita (expandida)	0.157
Aserrín o viruta	0.154

*Valores del ASHRAE Handbook of Fundamentals, 1972, American Society of Heating, Refrigerating, and Air-Conditioning Engineers, Nueva York.

El espesor de los materiales aislantes puede calcularse de acuerdo con los valores de la Tabla 4.3.5. Se supone que la velocidad del viento es menor de 24 km/h. El espesor del aislante debe aumentarse a mayores velocidades del viento. La resistencia térmica de los aislantes se ha calculado suponiendo que se aplica a la superficie externa de las cimbras de acero. Cuando se utilizan cimbras de triplay de 20 mm (¾"), la resistencia térmica, *R*, de los aislantes, que se proporciona en las tablas y curvas, puede aumentarse en 0.176°C·W·m², de acuerdo con la Tabla 4.3.5. Dado que las esquinas y las orillas son particularmente vulnerables, el espesor del aislante para estas partes debe ser aproximadamente tres veces el espesor requerido para muros o losas. Para cemento del Tipo II: los requerimientos para aislante que se proporcionan en las tablas deebn aumentarse aproximadamente un 30%. Cuando se utilicen otros tipos de cemento, deben realizarse ajustes similares en proporción con la cantidad de aislante empleado.* No es aconsejable una cantidad de aislante superior a la requerida, puesto que puede elevar la temperatura interna del concreto más allá del nivel recomendado, alargando, así, el periodo gradual de enfriamiento, e incrementando la contracción térmica, al igual que el riesgo de agrietamiento debido al shock térmico.

4.4 Aunque resultan más caros que otros medios de protección, los recintos cerrados son probablemente los más efectivos. Impiden el paso del viento, evitan la entrada de aire frío y conservan el calor. Se pueden hacer de cualquier material adecuado tal como madera, lona, yeso prefabricado o película de plástico. Los recintos fabricados de materiales flexibles son más baratos y más fáciles de poner y remover. Los recintos fabricados de materiales rígidos son más efectivos para impedir el paso del viento y mantener las temperaturas del perímetro. Los recintos deben soportar las cargas de viento y de nieve, y ser razonablemente herméticos. Debe proporcionarse el espacio suficiente entre el concreto y el recinto para permitir la libre circulación del aire caliente. Si se proporciona la altura suficiente, los obreros pueden trabajar con mayor eficiencia.

Por medio de vapor, aire caliente forzado y calentadores estacionarios o de otros tipos se puede suministrar calor a los recintos.

Aunque el calentamiento por medio de vapor proporciona un medio ambiente ideal para el curado, las condiciones de trabajo que ofrece distan mucho de ser las ideales y puede provocar problemas de hielo en los alrededores del perímetro del recinto.

Los ventiladores para aire forzado, que se calientan con petróleo o con gas, son, tal vez, las fuentes de calor más populares, ya que el aire caliente circula alrededor del concreto. Los gases de escape deben ventilarse hacia el exterior con el fin de evitar daños por el ataque del bióxido de carbono a las superficies del concreto fresco expuesto. Idealmente, estos calentadores deben estar colocados fuera del recinto, el cual debe recibir el aire caliente forzado a través de ductos. Los calentadores también pueden funcionar por medio de electricidad. Estos calentadores, al igual que los estacionarios, producen calor seco, por lo tanto, debe tenerse cuidado con objeto de evitar que se sequen las superficies de concreto expuesto. Esa deshidratación puede evitarse cubriendo el concreto con capas de material impermeable.

Los fuegos directos, o los calentadores estacionarios de petróleo o de carbón, tales como los braseros o salamandras, deben evitarse aun en obras pequeñas. No proporcionan la circulación de aire que se puede obtener con los chorros de vapor o los ventiladores y la adecuada ventilación de los gases de desecho resulta punto menos que imposible de conseguir. El peligro de incendio con este tipo de calentador es especialmente grande. En todo momento en la obra debe estar disponible un equipo contra incendios adecuado, a fin de garantizar la protección contra este riesgo. Asimismo, en todo momento debe haber personal presente que mantenga en constante operación las unidades de calentamiento.

* En "Concrete for Massive Structures", *Bulletin* No. IS128T, Portland Cement Association, Skokie, se pueden encontrar curvas típicas de calor de hidratación para diversos cementos.

4.5 El calentamiento interno del concreto se puede lograr ahogando en él resistencias eléctricas en espiral aisladas. Se pasa una corriente de bajo voltaje a través de las espirales ahogadas en la masa, cerca de la superficie de la sección, de acuerdo con un patrón predeterminado. La temperatura interna del concreto se puede elevar a cualquier nivel que se requiera disminuyendo el espaciamiento o la pendiente de las espirales, y el enfriamiento progresivo se puede controlar con interrupciones intermitentes de la corriente que pasa a través de ellas. El calentamiento se inicia, por lo general, después de un periodo de prefraguado de entre 4 y 5 horas, dependiendo de las características de fraguado del concreto. De esta manera, las cimbras aislantes se requieren únicamente para evitar el congelamiento del concreto durante el periodo de prefraguado y para minimizar la disipación del calor de las superficies que carecen de espirales. La pérdida de humedad en las superficies sin cimbra debida a la evaporación debe evitarse cubriéndolas con capas de plástico. Las temperaturas del concreto deben vigilarse por medio de dispositivos de control con objeto de que no se excedan las temperaturas recomendadas.

4.6 Durante la colocación del concreto, las lonas o cualquier otro tipo de cubiertas fácilmente movibles, extendidas sobre caballetes o armazones, deben seguir estrechamente el proceso de colocación, de manera que únicamente unos cuantos metros de concreto queden expuestos a la intemperie en cualquier momento. Dichas lonas deben arreglarse de modo que el aire caliente pueda circular libremente, tanto en la parte superior como, exceptuando los pavimentos, en la parte inferior de la losa. Para la prevención del congelamiento, también resultan efectivas las capas de material aislante colocadas directamente sobre el concreto, envolviéndolo. Esta protección es particularmente importante en el caso del concreto ligero estructural, debido a que su capacidad de mayor retención de calor permite un congelamiento más rápido de las superficies, que el concreto de peso normal.

4.7 Las envolturas y los recintos deben permanecer colocados durante todo el periodo de protección recomendado. Las secciones se pueden remover temporalmente con objeto de permitir la colocación adicional de cimbras o de concreto, pero la programación de este trabajo debe garantizar que no se permitirá la congelación de dicho concreto.

Estas secciones deben reemplazarse tan pronto como la cimbra o el concreto han tomado su posición definitiva. El tiempo perdido en el periodo de protección requerido debe recuperarse a la temperatura requerida, antes de retirar la protección.

4.8 Cuando se utilizan cimbras aislantes junto con recintos calentados, es aconsejable vigilar la temperatura, tanto del interior como de la superficie del concreto, por medio de dispositivos de control, con el fin de asegurarse de que el concreto, especialmente el concreto masivo, no se caliente más de lo necesario.

CAPITULO 5

métodos y requisitos para el curado en clima frío

5.1 Con objeto de que alcance la hidratación adecuada, el concreto nuevo debe estar protegido del secado prematuro. Por lo general, deben tomarse medidas positivas a fin de evitar una evaporación excesiva de humedad de dicho concreto. No obstante, durante el invierno, cuando la temperatura del aire cae por debajo de los 10°C, las condiciones atmosféricas en la mayor parte de las zonas no provocarán un secado indeseable; pero el concreto nuevo, en condiciones de saturación, resulta vulnerable al congelamiento y, por lo tanto, debe permitírsele un ligero secado antes de exponerlo a temperaturas de congelamiento.

5.2 A pesar de que el concreto expuesto al clima frío no es susceptible de secarse a una rapidez no deseada, debe prestarse especial atención a aquel concreto que sí está protegido, según se especifica en la Tabla 1.4.2. En tanto las cimbras permanezcan en su lugar, las superficies adyacentes a éstas estarán curadas adecuadamente en clima frío. Pero, las superficies sin cimbra, en particular los pisos terminados, tienden al secado rápido en un recinto cerrado. Cuando un concreto es calentado a más de 16°C y expuesto a una temperatura del aire de 10°C, es indispensable que se tomen medidas positivas a fin de evitar el secado. La técnica preferida consiste en utilizar vapor, tanto para el calentamiento, como para evitar la evporación excesiva. Si se va a utilizar un compuesto líquido para el curado del tipo formador de membrana, éste no debe aplicarse sino hasta que se ha terminado el uso del vapor. Cuando se utiliza un calor seco, el concreto debe estar cubierto con un material impermeable, o un compuesto para "curado" que cumpla con los requisitos de la norma ASTM C-309, o se puede curar con agua. El curado con agua es el método menos deseable, dado que, en clima extremadamente frio, ocasiona problemas de formación de hielo donde el agua se filtra de los recintos o donde existe un sellado deficiente. Asimismo, incrementa la posibilidad de que el concreto se congele en condiciones próximas a la saturación, una vez que se remueve la protección.

5.3 Cuando la temperatura del aire ha caido a 10°C, el concreto puede quedar expuesto al aire dentro del recinto, siempre y cuando la humedad relativa no sea inferior al 40%. Cuando el clima es extremadamente frio, siempre resulta necesario añadir humedad al aire calentado con objeto de mantener esta humedad. Por ejemplo, si la temperatura exterior es de —12°C, la humedad relativa dentro del recinto calentado será inferior al 20% si no se añade humedad. Si se utiliza el curado con agua, éste debe darse por terminado 12 horas antes del periodo de protección de temperatura, y debe permitirse que el concreto seque antes y durante el periodo de ajuste gradual a las condiciones de clima frio ambiental que se mencionan en la linea 5 de la Tabla 1.4.1.

5.4 Una vez removida la protección para conservar la temperatura, no se requiere medida positiva alguna para evitar la evaporación excesiva, en tanto la temperatura del aire permanezca por debajo de los 10°C. Una excepción a esto la constituye el concreto colocado en regiones extremadamente áridas. Cuando el concreto a 10°C queda expuesto al aire con una temperatura de 10°C, y una humedad relativa menor al 40%, o al aire con una temperatura de 5°C y una humedad relativa menor al 60%, el secado será excesivo. Cuando la temperatura del concreto ha caido a 5°C, una temperatura ambiente de 5°C, con una humedad relativa del 11% puede ser tolerada. Si se espera un secado excesivo y no se espera congelamiento, se puede curar el concreto con agua. De lo contrario, el procedimiento preferido es el empleo de compuestos para "curado" o de una cubierta permeable. A temperaturas superiores a los 10°C, el secado aumenta con rapidez. Durante el periodo invernal, en el cual se presenta el congelamiento, las temperaturas extremas ocasionales, superiores a los 10°C, no deben ser motivo de preocupación. Sin embargo, cuando se presentan temperaturas superiores a los 10°C durante más de la mitad de un periodo de 24 horas, ya no se debe considerar al concreto como concreto de invierno y debe aplicarse una práctica de curado normal.

5.5 A pesar de que el concreto expuesto a clima invernal no se seca a una velocidad indeseable, la aplicación de un compuesto para "curado" reduce el secado y, por lo tanto, mejora las condiciones de curado. Si se aplica un compuesto durante el primer periodo de temperatura superior al punto de congelación después de que se ha removido la protección, se elimina la necesidad de efectuar operaciones adicionales de curado si la temperatura se llegara a elevar por sobre los 10°C.

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CAPITULO 6

requisitos de protección para el concreto bajo otras condiciones de servicio aparte del estructural

6.1 Para concreto que no sea estructural, el cual se describe en el Capitulo 5, los requisitos de protección necesarios en cada una de las tres condiciones generales de servicio son los siguientes:

6.1.1 La primera incluye renglones tales como cimientos y subestructuras que no estarán sujetos a una carga a temprana edad y que, debido a su localización profunda o rellenada, sufrirán poco o ningún congelamiento y deshielo en el servicio. En estas circunstancias de construcción y servicio, las condiciones favorecerán el curado natural continuado. Este concreto únicamente requeriria el tiempo de protección recomendado para la primera condición en la Tabla 1.4.2.

6.1.2 La segunda incluye concreto en pilas masivas y presas que contarán con superficies expuestas al congelamiento y a la acción del clima durante el servicio, pero que no tendrán requisitos de resistencia a temprana edad. Las porciones interiores serán autocurables. Las superficies exteriores continuarán curándose cuando las condiciones naturales sean favorables. Con objeto de proporcionar un curado inicial y de asegurar la durabilidad de las superficies y de las orillas durante el servicio, este concreto debe recibir, cuando menos, la cantidad de protección recomendada para la segunda condición en la Tabla 1.4.2. El concreto en estructuras masivas, en las cuales el incremento de temperatura interna es critico, no debe sufrir aceleración en su desarrollo de resistencia.

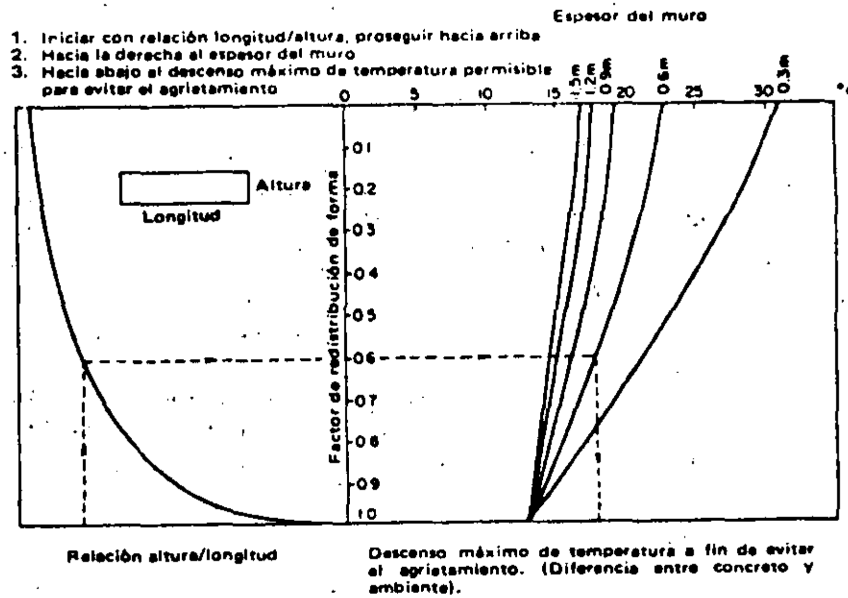
6.1.3 La tercera condición incluye estructuras expuestas a la acción del clima, que pueden estar parcialmente cargadas antes de que las condiciones de curado natural hayan desarrollado plenamente la resistencia del concreto. Un concreto tal debe contar, por lo menos, con la cantidad de protección que para la tercera condición se recomienda en la Tabla 1.4.2. Cuando las condiciones que se esperan en la obra respecto a la carga, las condiciones climáticas, los materiales y la mezcla probable, son conocidas, se puede estimar la confirmación de la cantidad de protección y de curado que se requieren a 10°C, en base a los datos apropiados (referencias 1 y 13) de no hacerse especimenes de prueba curados en la obra.

6.2 Durante la época de frio, con frecuencia es altamente significativa la protección que proporcionan las cimbras que no son de acero. En recintos calentados, las cimbras sirven para distribuir el calor más equitativamente y para evitar el calor concentrado. Con el aislante debido, en muchas ocasiones, las cimbras, incluyendo las de acero, proporcionan una protección adecuada sin calentamiento suplementario. Por lo tanto, a menudo resulta ventajoso dejar las cimbras hasta que termine el periodo de protección mínimo requerido, o aún más tiempo. No obstante, el programa de construcción o la reutilización, por cuestión de economía, de las cimbras, con frecuencia determinarán su remoción a la brevedad posible. En este caso, las cimbras se pueden remover tan pronto como esto se pueda realizar sin causar daño o poner en peligro al concreto. Si se utilizan cuñas para separar la cimbra del concreto joven, éstas deben ser de madera. Por lo general, cuando el concreto ya es lo suficientemente resistente, las esquinas y las orillas no sufrirán daños durante el descimbrado. El tiempo mínimo necesario para esto es preferible determinarlo de manera tenativa en la obra misma, ya que influyen en él diversos factores propios de la obra en sí, los cuales incluyen el tipo y la cantidad de cemento, al igual que otros aspectos de la mezcla del concreto, la temperatura de curado, el tipo de la estructura, el diseño de las cimbras y la habilidad de los trabajadores. Generalmente, para entonces ya será lo suficientemente resistente como para soportar su propio peso sin que se presente agrietamiento. Después de la remoción de la cimbra, el concreto debe cubrirse con mantas aislantes o mantenerlo encerrado en recintos calentados durante el tiempo recomendado en la Tabla 1.4.2. Si se utiliza calor interno por medio de resistencias ahogadas en espiral, el concreto debe cubrirse con capas impermeables y debe continuarse el calentamiento durante el tiempo recomendado.

6.3 En el caso de muros de contención, muros de cimentación u otras estructuras, en las cuales únicamente uno de los lados estará sujeto a presión hidrostática, la apresurada remoción de la cimbra, mientras el concreto es aún relativamente joven, puede dislocar los amarres y separadores de la cimbra y formar canales por los cuales puede fluir el agua.

6.4 Al llegar a su fin el periodo de protección, el concreto debe

enfriarse gradualmente de acuerdo con lo indicado en la Tabla 1.4.1. Esto se puede lograr con facilidad, reduciendo el calor que proviene de fuentes externas. No obstante, las cimbras aislantes pueden presentar algunas dificultades. Puede resultar satisfactorio abrir un poco las cimbras y cubrirlas con capas de polietileno, a fin de permitir una ligera circulación del aire.



6.5 Aunque el concreto debe enfriarse gradualmente hasta que esté a temperatura ambiente, a fin de evitar el agrietamiento térmico, se puede permitir un diferencial de temperatura entre el concreto y la temperatura ambiente en el momento de suspender la protección. La diferencia máxima permisible entre el concreto y el aire ambiental (si los vientos no exceden de 24 km/h) se puede determinar por medio de la Fig. 6-5. Las curvas ofrecen compensación por el espesor del muro y la restricción del factor de forma.

CAPITULO 7

requisitos de protección para el concreto estrcutural

7.1 Los criterios para el descimbrado del concreto estructural deben basarse en la resistencia del concreto en la obra y no en una duración arbitraria de tiempo. Para el concreto estructural, el cual debe alcanzar un considerable nivel de resistencia de diseño, previo a un descimbrado y un desapuntalamiente finales seguros, debe proporcionarse un tiempo de protección adicional superior al mínimo establecido en la Tabla 1.4.2, dado que este mínimo únicamente sería suficiente desde el punto de vista de la resistencia al daño por congelamiento.

7.2 Un método utilizado para verificar el logro de la suficiente resistencia antes de reducir, cambiar o remover el apoyo, y antes de suspender el curado y la protección, consiste en elaborar, por lo menos, seis cilindros de prueba en la obra, tomados de los últimos 75m³ de concreto, de los cuales, por lo menos 3 deben elaborarse por cada 2 horas durante toda la etapa de colocación del concreto, o por cada 75 m³, según el que proporcione el mayor número de cilindros. Dichos especímenes deben hacerse de acuerdo con la norma ASTM C 31 y, de inmediato, deben protegerse del clima frío, en tanto puedan colocarse bajo la misma protección proporcionada a las partes de la estructura a las cuales representan. Después de retirarlos del molde debe procederse al cabeceado y prueba de los especímenes, de acuerdo con las secciones adecuadas de las normas ASTM C 31 y C 39. Las pruebas de resistencia a la compresión del concreto y las no destructivas efectuadas en la obra, cuando están correlacionadas con los resultados de la prueba de cilindros con

18
47

curado normal y curado en la obra (ASTM C 192), constituyen un método adicional que se puede utilizar con el fin de verificar el logro de la resistencia.

7.3 Puesto que la ganancia de resistencia del concreto es una función del tiempo y de la temperatura, el cálculo del desarrollo de la resistencia del concreto en una estructura también se puede llevar a cabo relacionando los incrementos de temperatura y tiempo del concreto en la obra con la resistencia de los cilindros de la misma mezcla de concreto, curados bajo condiciones estándar de laboratorio. Esta relación ha sido establecida[18] por medio del uso del factor de madurez, M, expresado como:

$$M = \Sigma(C + 10)\Delta t$$

donde

C = temperatura en °C

Δt = duración del curado a la temperatura C, en horas o días.

La ecuación supone que la resistencia no aumenta a temperaturas inferiores a aproximadamente −10°C. En el concepto de madurez, el desarrollo de la resistencia está considerado como equivalente a la suma de las contribuciones de cada periodo de curado. Cuando la temperatura es constante, como en los métodos de curado en el laboratorio, el signo de adición no es necesario.

7.3.1 La curva del factor de madurez-resistencia se establece efectuando pruebas de resistencia a la compresión, a diversas edades, a una serie de cilindros de concreto similar al cual se va a utilizar en la obra, curados en el laboratorio, a una temperatura estándar (23 ± 1.7°C).

La verdadera resistencia del concreto colocado en un sitio en particular, en un momento en particular, puede predecirse por medio de la determinación de su factor de madurez en el momento dado y la lectura de la resistencia correspondiente en la curva factor de madurez-resistencia.

El factor de madurez del concreto en la obra se determina midiendo la temperatura del concreto, en el sitio en particular, a periodos de tiempo reducidamente separados y sumando los productos sucesivos de los periodos de tiempo y la temperatura promedio del concreto durante ese periodo.

7.3.2 La siguiente información debe estar disponible de modo que se pueda llevar a cabo el cálculo de la resistencia del concreto en la obra:

a) La relación del factor de madurez-resistencia del concreto que se está colando bajo condiciones estándar de laboratorio.

b) Un registro de temperatura-tiempo del concreto en la obra. Estos se pueden obtener por medio del uso de termistores o termopares colados a diversas profundidades en el concreto. La ubicación que arroja los más bajos valores debe ser la serie de temperaturas utilizada en los cálculos.

c) Se pueden obtener "Medidores de madurez del concreto", los cuales permiten la determinación continua y directa del factor de madurez, en un lugar en particular del concreto en la obra. Los medidores utilizan una sonda ahogada en el concreto, la cual mide, de manera continua, la temperatura; automáticamente la integra al tiempo, y continuamente muestra el factor de madurez en un contador marcado en grados Celsius-horas.

7.3.3 *Cálculos de ejemplo.* Previniendo un periodo de clima frío, un contratista instaló termopares en un muro de concreto, colado el 1° de septiembre de 1976, a las 9:00 a.m. Se había establecido una curva factor de madurez-resistencia y una historia de la ganancia de resistencia, bajo condiciones de laboratorio, se había desarrollado de la mezcla de concreto utilizada (Fig. 7.3.3). Se mantuvo un registro de las temperaturas del concreto en la obra. A los tres días (72 horas) el contratista necesitaba saber la resistencia del concreto en la obra. Si utiliza el registro de temperatura, tal como se indica en las columnas 3, 4 y 5 de la Tabla 7.3.3, con objeto de determinar el factor de madurez equivalente que se muestra en la columna 9, le resulta posible determinar que la resistencia del concreto colocado es aproximadamente de 126 kg/cm², tomando como base la curva factor de madurez-resistencia.

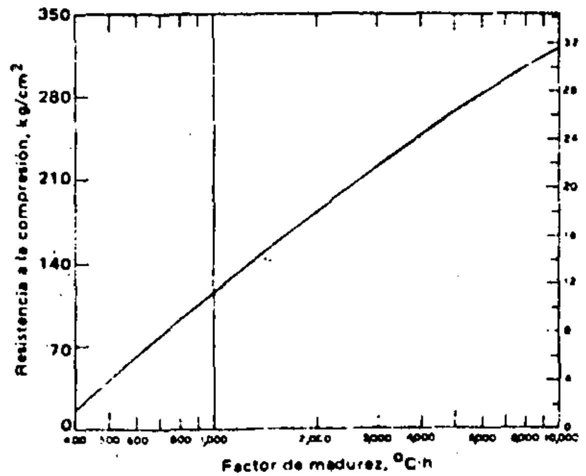


Figura 7.3.3.– Relación del factor resistencia/madurez para especímenes curados en el laboratorio (22.8°C)

TABLA 7.3.3.– CALCULO DEL FACTOR DE MADUREZ Y PREDICCION DE RESISTENCIA DEL CONCRETO EN EL LUGAR

1	2	3	4	5	6	7	8	9
Fecha	Tiempo transcurrido hr	Temperatura en la estructura °C	Temperatura promedio en la estructura °C	(Col. 4) + 10°C °C	Intervalo de tiempo hr	Col. 5 x Col. 6, °Chr	Factor de madurez Σ Col. 7 °Chr	Resistencia a la compresión correspondiente, kg/cm²
Sept. 1	0	10						
			10	—	—	—	—	—
	12	10						
			10	20	12	240	240	--
2	24	10						
			9	19	6	114	354	—
	30	8						
			8.5	18.5	18	333	687	76
3	48	9						
			8.5	18.5	12	222	909	105
	60	8						
			7.5	17.5	12	210	1119	122
4	72	7						
			6.5	16.5	96	1584	2703	208
8	168	5						
			6	16	72	1152	3855	239
11	240	6						
			6	16	72	1152	5007	262
14	312	6						

Nota: Temperatura de curado en el laboratorio = 22.8°C
Supóngase que no hay ganancia de resistencia 0 – 12 hr
Cálculos basados en un punto de temperatura localizado a 10 cm por debajo del lecho superior de la losa de azotea.

7.4 En términos generales, son escasas las oportunidades de proporcionar un curado adicional al concreto estructural aparte del proporcionado inicialmente. En la Fig. 7.4 se ilustra el efecto en la resistencia de suspender el curado húmedo y de exponer al concreto al aire, a diversas edades. Por esta razón, las resistencias tempranas, lo suficientemente altas para asegurar la obtención de la resistencia de diseño, deben ser logradas antes de que el concreto estructural, con apoyos temporales, pueda ser librado de la protección invernal y expuesto a temperaturas de congelamiento.

7.5 El tiempo que el concreto requiere para lograr la resistencia

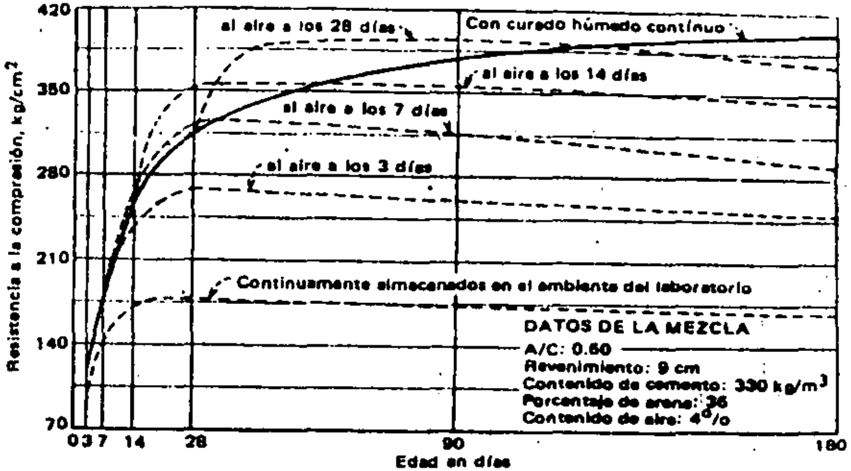


Figura 7.4.– Resistencia a la compresión de un concreto secado al ambiente del laboratorio, después de un curado húmedo preliminar.[19] (Especímenes probados después de secados al aire).

requerida y la segura remoción de los puntales se ve influido por muchos factores. Entre éstos, los más importantes son aquellos que afectan la rapidez y el nivel del desarrollo de la resistencia, tales como la temperatura inicial del concreto en el momento de la colocación, la temperatura a la cual se mantiene el concreto después de la colocación, el tipo del cemento, el tipo y la cantidad del aditivo acelerante o de otros aditivos utilizados y las condiciones de protección y curado. La duración de la protección puede reducirse de manera considerable por medio de:

7.5.1 Mantener la temperatura durante la protección y curado a un nivel más alto después del colado que el indicado en la línea 1 de la Tabla 1.4.1. Véase la Fig. 7.5.1. Los cementos del Tipo I y del Tipo III proporcionan resistencias algo más altas que el Tipo II a edades tempranas. Con motivo de las variaciones en el comportamiento de cualquier cemento dado, los datos proporcionados en la Fig. 7.5.1 deben utilizarse únicamente como guía.

7.5.2 Utilizar tipos y composiciones de cemento que demuestren un desarrollo de resistencia más temprano. Véase la sección 8.1.

7.5.3 Utilizar un aditivo acelerante, tal como el cloruro de calcio, o uno que esté de acuerdo con la norma ASTM C 494, del Tipo C (acelerante) o E (reductor de agua y acelerante). No obstante, deben revisarse las diversas precauciones que se mencionan en el Capítulo 8 antes de utilizar cloruro de calcio o aditivos del Tipo C, o del Tipo E que contienen cloruro de calcio. Debido a las variaciones en el comporta-

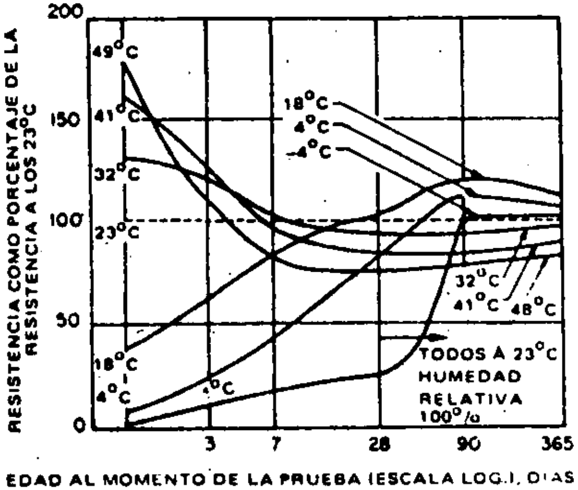


Figura 7.5.1.— Efecto de las condiciones de temperatura en el desarrollo de la resistencia del concreto (Cemento Tipo I)[13]

miento según las distintas marcas y tipos de cemento, deben efectuarse pruebas previas a la temperatura de curado prevista, utilizando el cemento, los agregados y los aditivos propuestos.

7.6 Deebn tomarse las precauciones adecuadas a fin de asegurar el enfriamiento gradual del concreto al llegar a su fin el periodo de proteccion.

7.7 Ocasionalmente es posible que sea necesario tomar decisiones cuando no se actuó de la maneda adecuada, a fin de confirmar la suficiencia del curado y de la protección para producir niveles seguros de desarrollo de resistencia. Para concretos que de ordinario alcanzarían un margen seguro de resistencia promedio, superior a la resistencia de diseño, f'_c, con 28 dias de curado húmedo estándar a 23°C, la Tabla 7.7 ilustra el número aproximado de dias de curado y protección recomendados a 10 y a 21°C, con objeto de obtener el porcentaje de resistencia de diseño que requiere el ingeniero o el arquitecto para la remoción segura de las cimbras y los puntales. El número de dias y el porcentaje de f'_c ilustrados en la Tabla 7.7, reflejan la velocidad a la cual resulta posible que el concreto logre la f'_c requerida durante la exposición indicada. Sin embargo, en previsión de este posible problema, el ingeniero debe desarrollar una curva factor de madurez-resistencia, basada en los materiales específicos utilizados en la obra.

TABLA 7.7.— DURACION DE LA PROTECCION RECOMENDADA PARA EL PORCENTAJE DE RESISTENCIA REQUERIDO

A 10°C, días				A 21°C, días			
Porcentaje de la resistencia de diseño, f'_c requerida	Tipo de cemento			Porcentaje de la resistencia de diseño, f'_c requerida	Tipo de cemento		
	I	II	III		I	II	III
50	6	9	3	50	4	6	3
65	11	14	5	65	8	10	4
85	21	28	16	85	16	18	12
95	29	35	26	95	23	24	20

Los datos de esta tabia proceden de concretos con resistencias de 210 a 350 kg/cm² después de 28 días de curado a 21 ± 1.7 °C. Se consideró esta resistencia a los 28 días, para cada tipo de cemento, como el 100°/o en la determinación del tiempo necesario para alcanzar diversos porcentajes de esta resistencia, 10°C y con 21°C durante el curado. Por necesidad, estos valores son sólo aproximados y promedio. Deben obtenerse los valores específicos con materiales y mezclas de la obra.

7.8 Las recomendaciones que se dan en este Capitulo y la Tabla 7.7 se basan en disposiciones o en la existencia de ciertas condiciones de la obra, que cumplen con las normas minimas y requerimientos básicos siguientes:

7.8.1 La temperatura interna del concreto debe estar, por lo menos, a 10°C después de la colocación de dicho concreto. A fin de mantener al mínimo la contracción termica subsecuente, debe tratarse de que esta temperatura se exceda lo menos posible en cualqúier época del año.

7.8.2 Las instalaciones deben ser capaces de mantener la temperatura del concreto, durante todo el proceso, a 10°C, o más, hasta que se pueda garantizar que esta protección se puede suspender con seguridad. Dichas instalaciones incluyen:

a) Abrigo adecuado y proteccion contra el viento y la perdida de calor.

b) Equipo efectivo y personal suficiente con objeto de mantener todas las partes del concreto a la temperatura requerida, al igual que el equipo de protección contra incendios necesario.

c) Protección y sistema de calentamiento que abarque la superficie superior de las nuevas losas o pisos, y que deje descubiertas las aberturas, a fin de proporcionar circulación de aire, a menos que se utilicen cubiertas aislantes.

d) Ventilación y circulación en la medida en que sean necesarias para mantener una temperatura constante en las partes superior e inferior de elementos verticales tales como muros, pilas y columnas.

7.8.3 Los puntales deben permanecer colocados el tiempo necesario para salvaguardar todos los miembros y la estructura. El número de hileras reapuntaladas por debajo de la que se está colando y el tiempo que debe permanecer colocado dicho reapuntalamiento deben basarse en evidencia confiable que se tenga de que la resistencia obtenida es la suficiente para soportar con seguridad el total de las cargas involucradas.

7.8.4 El concreto debe hacerse con cemento portland de los Tipos I, II y III.

7.8.5 Debe efectuarse un curado adecuado, en particular con el fin de evitar el secado en recintos calentados.

7.8.6 Debe llevarse a cabo una inspección con objeto de cumplir con los requisitos de construcción del Reglamento de las construcciones del ACI, al igual que con otras normas de inspección de dicho Instituto.

7.9 La remoción de cimbras y apoyos, al igual que la colocación y remoción de puntales debe estar de acuerdo con la norma ACI 347 "Recommended Practice for Concrete Formwork". Entre otros requisitos, la norma mencionada recomienda lo siguiente:

7.9.1 El ingeniero o el arquitecto debe determinar y especificar la resistencia requerida del concreto en la obra, con objeto de permitir la remoción de cimbras y de puntales.

7.9.2 Deben efectuarse pruebas adecuadas a los cilindros de concreto curados en la obra o del concreto en la obra (véase la sección 7.2).

7.9.3 En las especificaciones deben prescribirse, en su totalidad, los métodos para evaluar los resultados de prueba del concreto, al igual que la resistencia mínima requerida.

7.9.4 Deben registrarse y ser utilizados por el ingeniero o el arquitecto los resultados de todas las pruebas, al igual que los registros de las condiciones climáticas y cualquiera otra información pertinente, con objeto de tomar la decisión respecto a cuándo deben removerse las cimbras y los puntales.

7.9.5 El ingeniero debe planear de antemano y revisar cualquier proceso de reapuntalamiento, mismo que constituye una de las operaciones más críticas relacionadas con las cimbras. Esta operación debe efectuarse de manera que, en ningún momento se requiera que grandes áreas nuevas de construcción soporten cargas muertas y de construcción combinadas que sobrepasen la capacidad determinada por las cargas de diseño y el desarrollo de la resistencia del concreto en el momento del descimbrado y reapuntalamiento.

CAPITULO 8

aceleración del desarrollo de resistencia

8.1 Aspectos generales

Si se observan las precauciones adecuadas, se pueden utilizar aditivos acelerantes, cemento de alta resistencia a temprana edad (Tipo III) o cemento adicional, con el fin de desarrollar el nivel de resistencia requerido en un periodo más corto. Esta aceleración de ganancia de resistencia con frecuencia tiene como resultado un ahorro considerable debido a la menor duración de la protección, la facilidad de volver a utilizar la cimbra con mayor rapidez, la más temprana remoción de los puntales, o menos mano de obra para los acabados de los elementos planos (véase ACI 302). Cuando estos medios se utilizan con objeto de obtener una más alta resistencia a temprana edad, el incremento del calor de hidratación resultante puede ser favorable en algunas ocasiones. Los factores de composición del cemento, tales como un más alto contenido de silicato o aluminato tricálcico, cuando por cualquiera otra razón no son adversos, resultan ventajosos en clima frío con motivo de su contribución a un calor por hidratación más alto y más temprano. Sin embargo, aun los cementos del mismo tipo varían ampliamente entre sí en su relación tiempo-resistencia. Se recomienda efectuar pruebas con el cemento propuesto para usarse en la estructura, con objeto de demostrar cual de estas posibilidades será la más conveniente para este propósito en particular.

8.2 Cloruro de calcio como aditivo acelerante

8.2.1 El cloruro de calcio es un aditivo acelerante ampliamente utilizado y de gran popularidad. El uso y los efectos de dicho cloruro se describen en "Guia para el uso de los aditivos en el concreto" presentado por el Comité ACI 212 (traducida por el Instituto Mexicano del Cemento y del Concreto, A. C., junio, 1976, 84 pp.), al igual que en la referencia 3. No existen los suficientes datos acerca del uso del cloruro de calcio con cemento portland con escoria de alto horno, o con otros cementos combinados que justifiquen cualquier recomendación, positiva o negativa, respecto a su empleo con estos cementos. No obstante, en ciertas condiciones no se debe utilizar el cloruro de calcio, y las siguientes aplicaciones deben registrarse cuidadosamente.

a) No se debe utilizar en concreto presforzado debido a su potencial para aumentar la corrosión metálica a causa del esfuerzo.

b) La presencia de los cloruros se ha relacionado con la corrosión galvánica del acero galvanizado cuando este material se utiliza como cimbra permanente en cubiertas o en elementos empotrados, y en construcciones de este tipo no se recomienda el uso de cloruro de calcio adicional.

c) Asimismo, recientes estudios han indicado que la corrosión galvánica de los metales ahogados en el concreto se intensifica mediante la adición de cloruro de calcio al concreto. Los siguientes constituyen ejemplos de esto:

1) Cuando se conectaron externamente hojas de aluminio y de acero, ahogadas en el concreto, la corrosión galvánica del aluminio fue proporcional a la concentración de cloruro de calcio.

2) Donde se colaron grandes losas de concreto reforzado, en las cuales había conductos ahogados de aluminio y acero unidos eléctricamente, los resultados demostraron que el cloruro de calcio fomentaba la corrosión galvánica. (*Proceedings*, ASCE, Vol. 89, ST5, octubre de 1973, pp. 117-132.)

d) No debe utilizarse cloruro de calcio donde se requiere concreto resistente a los sulfatos; un 20% adicional de cemento resistente a los sulfatos resultará mucho más beneficioso, pues incrementará, en vez de disminuir, la resistencia del concreto a dichos sulfatos.

e) Si no se controla la reacción álcali-sílice del cemento, mediante el uso de un cemento con bajo contenido de álcalis o de una puzolana efectiva, el cloruro de calcio puede incrementar la expansión.

f) En las zonas de alta concentración de cloruros, las variaciones en el contenido de calcio del concreto en un elemento aislado, que son

provocadas por los cambios en las condiciones climáticas, pueden crear células de concentración y ocasionar una severa corrosión del acero de refuerzo con protección deficiente. Esta práctica es peligrosa y no debe ser permitida en las estructuras en las que existe continuidad en el acero.

g) Aparte de las condiciones arriba descritas, bajo las cuales no debe utilizarse el cloruro de calcio, una considerable cantidad de casos confirmados de refuerzo corroído y el consecuente deterioro de las estructuras de concreto que contienen cloruros han sido registrados donde no existía razón aparente para prohibir el cloruro de calcio. En vista de está peligrosa capacidad para ocasionar daños por corrosión cuando se utilizan cloruros, en cualquiera de sus formas, en cantidades significativas, cualquier aceleración que pudiera necesitarse debe obtenerse por otros medios, como antes ya se mencionó en este capítulo.

8.2.2 El cloruro de calcio u otros productos químicos en la mezcla, en cantidades permisibles, no reducirán el punto de congelación del concreto de manera significativa. A fin de evitar una confianza injustificada en este método y con objeto de evitar el uso de materiales dañinos, no debe permitirse un intento tal de proteger al concreto del congelamiento.

8.3 Otros aditivos acelerantes

8.3.1 Se ha encontrado que algunos acelerantes reductores de agua, según la norma de la ASTM C 494 "Chemical Admixtures for Concrete" (Aditivos químicos para el concreto), del Tipo E, aceleran la ganancia de resistencia, a temperaturas ambiente de 10°C e inferiores, y reducen el contenido unitario de agua de la mezcla. La mayoría de los aditivos del Tipo E contienen pequeños porcentajes de cloruro de calcio que, por lo general, no alcanzan el 0.2%, por peso, de cemento, cuando se utilizan en las proporciones de dosificación recomendadas (véase la sección 8.3.2). Esta cantidad por sí misma, ejercería un efecto sumamente limitado en las propiedades que proporcionan una resistencia temprana al concreto. No obstante, las pruebas indican que algunos acelerantes reductores de agua, que cumplen con los requisitos de la norma ASTM C 494 mejorarán sustancialmente la resistencia a las 24 horas, cuando el concreto que los contiene, mezclado a 10°C, está expuesto a temperaturas de 10°C. La resistencia de un concreto como éste se aproximará a la que es posible obtener, mediante el 2% de cloruro de calcio y será superior a aquélla que se obtiene con algunos, pero no todos, los cementos del Tipo III. Los datos también indican que los acelerantes reductores de agua pueden producir considerables incrementos en la resistencia a cualquier edad posterior. En la norma ACI 212 "Guia para el uso de aditivos para el concreto"; en la ACI 201, "Durabilidad del concreto en servicio", y en el simposio patrocinado por el Comité ACI 222, SP-49, "Corrosión de metales en el concreto", se puede encontrar información adicional sobre este tema.

8.3.2 Si no se cuenta con la información adecuada, o no están disponibles los registros de comportamiento anteriores, deben llevarse a cabo pruebas con el fin de evaluar el efecto de un aditivo en particular sobre las propiedades del concreto de la obra, utilizando materiales semejantes y a las temperaturas esperadas en ésta, y apegándose a los procedimientos de construcción que se siguen en ella. Los fabricantes de muchos aditivos de patente anuncian propiedades de endurecimiento y desarrollo de resistencia acelerados. Ya que la mayoría de estos productos contienen cloruro de calcio, el usuario debe determinar si el aditivo en cuestión contiene cloruro de calcio o no, y, de ser así, el porcentaje, por peso del cemento, que su uso introducirá en el concreto. De acuerdo con esto, el potencial de corrosión debe entonces ser evaluado.

CAPITULO 9

registros de temperat ra

9.1 El personal a cargo de la inspección debe mantener registros de la fecha, la hora, la temperatura del aire exterior, la temperatura del concreto al momento de la colocación y de las condiciones climáticas (calmado, ventoso, despejado, nublado, etc.). El registro debe incluir la temperatura de diversos puntos dentro del recinto y en la superficie, las esquinas y las orillas del concreto, en cantidad suficiente para ilustrar el rango de temperaturas del concreto. Los dispositivos para medir la temperatura empotrados en el concreto son ideales, pero se puede obtener una exactitud satisfactoria y una mayor flexibilidad de observación si se coloca el termómetro contra el concreto bajo una cubierta temporal de material aislante pesado, hasta que registre una temperatura constante. También se pueden utilizar resistencias térmicas o termopares hables, colados en el concreto. En el curso de cada período de 24 horas, deben registrarse las lecturas de la temperatura máxima y de la mínima. Los datos registrados deben mostrar con toda claridad el desarrollo de la temperatura de cada parte del concreto. En los registros permanentes de la obra se debe incluir una copia de las lecturas de temperatura. Resulta preferible medir la temperatura del concreto en más de un lugar en cada miembro y utilizar la lectura más baja como representativa del promedio a lo largo de determinado período. Debe vigilarse la temperatura del concreto por medio de dispositivos de control, a fin de evitar un calentamiento excesivo.

CAPITULO 10

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- PRODUCCIÓN DE GRANDES VOLÚMENES DE CONCRETO

Colocación de concreto por medio de bandas transportadoras

(ACI-304)

INSTITUTO MEXICANO DEL CEMENTO Y DEL CONCRETO, A. C.

COLOCACION DE CONCRETO POR MEDIO DE BANDAS TRANSPORTADORAS

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Contenido

Sinopsis

Este reporte del estado actual de conocimientos incluye una historia del desarrollo de las primeras bandas transportadoras utilizadas para el traslado y la colocación de concreto. Presenta una discusión sobre el diseño de los sistemas de transportación, en relación a las propiedades del concreto en estado plástico, a la velocidad de colocación y a las especificaciones del trabajo. Considera el ancho de la banda, su velocidad y los ángulos de inclinación con respecto a los requerimientos específicos. Abarca los tres tipos de transportadores de concreto: los portátiles, los de alimentación y los de distribución, y sus aplicaciones particulares. Asimismo describe los métodos prácticos para la elección, uso y mantenimiento de los transportadores. También discute el ahorro que representa usar bandas transportadoras en la colocación del concreto. Finalmente, enfatiza la calidad del concreto colocado así como los procedimientos de inspección.

CAPITULO 1

Introducción

Las bandas transportadoras de concreto están diseñadas o modificadas especialmente para transportar concreto en estado plástico desde una fuente de suministro hasta las cimbras u otros lugares, sin tener que usar equipo adicional, excepto el requerido para la compactación. La colocación del concreto por medio de bandas transportadoras debe ser una operación continua. Los mejores resultados se obtienen cuando se cuenta con un suministro constante de concreto mezclado adecuadamente para cargar la banda transportadora, así como con dispositivos necesarios para desplazar el punto de descarga durante la colocación, de manera que el concreto en estado plástico pueda ser depositado en toda el area, sin necesidad de traspalearlo o aplicarle demasiada vibración.

Las bandas transportadoras se clasifican en tres tipos: (1) portátiles o autocontenidas, (2) de alimentación o en serie y (3) de distribución, con descarga radial o lateral.

Las primeras bandas transportadoras en Norteamérica se utilizaron para manejar [illegible]. En la "Millers Guide" de 1795 se describe una banda transportadora de [illegible]. La primera noticia que se tiene de una banda transportadora usada para manejar material más pesado que el grano proviene apenas de los años 1890s, cuando se instalaron algunas bandas transportadoras en una planta procesadora de mineral en Edison, N.J. La introducción comercial de cojinetes antifriccionantes en la construcción de rodillos guía abrió el camino para las bandas transportadoras modernas. En 1923 se emplearon por primera vez con éxito en el manejo de carbón.[1]

Según se sabe, la primera vez que se usaron las bandas transportadoras de concreto fue en 1929, cuando la Corbetta Construction Co., Inc. usó una banda transportadora de 183 m de longitud para colocar el concreto estructural del puente de la 238a. Calle Este, en el condado de Bronx, ciudad de Nueva York. La mezcla de concreto (1:2:4) contenía agregado de tamaño máximo de 19 mm.

Entre 19[illegible] y 1944, se utilizaron bandas transportadoras para trasladar concreto entre la planta mezcladora y un punto de distribución central, en donde era cargado en cucharones con descarga inferior para ser colocado en diversas obras del Corps of Engineers y de la TVA. Estos proyectos utilizaban de 190 a 256 kg de cemento por m³ de concreto y agregado de 100 a 150 mm de tamaño máximo. La segregación del agregado de mayor tamaño en los puntos de transferencia y en las tolvas era causa de problemas considerables; de aquí que se desarrollaran varios deflectores, canales y tolvas para reducir la segregación al mínimo.[2] De 1941 a 1950, la Ontario Hydro (Fig. 1) usó con éxito las bandas transportadoras para colocar concreto en siete diferentes construcciones de presas.[3] La Tabla 1 muestra las principales características del concreto manejado en estas operaciones.

TABLA 1.– PRIMERAS PRESAS EN DONDE SE MANEJO EL CONCRETO POR MEDIO DE BANDAS TRANSPORTADORAS

NOMBRE	PROPIETARIO O AGENCIA	CEMENTO, KG/M³, EN CORAZONES DE CONCRETO	AGREGADO, DE TAMAÑO MAXIMO, mm	PROPORCION DE AGREGADO FINO EN EL TOTAL DE AGREGADO, PORCENTAJE
Pine Canyon	Corps of Engineers	212	152	*
Mahoning Creek	Corps of Engineers	190	152	*
Tygart	Corps of Engineers	195	102	*
Fontana	TVA	190	152	30
Ft. Loudoun	TVA	196	127	25
Guntersville	TVA	256	102	27
Aquasabon	Ontario Hydro	182	76	42
Barrett Chute	Ontario Hydro	208	38	45.5
Des Joachims	Ontario Hydro	217	76	4[illegible].5
Otto Holden	Ontario Hydro	217	76	36
Pine Portage	Ontario Hydro	228	76	43
G. W. Rayner	Ontario Hydro	199	76	43
Stewartville	Ontario Hydro	197	76	43

*Datos no disponibles

Fig. 1.– Sistema estacionario de transporte en la Presa Des Joachims, construida por la compañía Ontario Hydro.

A principios de la década de los 50s, la disponibilidad de concreto premezclado para llevar a cabo proyectos de construcción en los Estados Unidos creó una gran demanda de equipo que facilitara librar el tramo entre el área accesible a un camión-mezcladora y el lugar donde debía ser colocado el concreto. A fines de dicha década llegaron al mercado las primeras bandas transportadoras de concreto portátiles, comercialmente accesibles.[4]

El transportador por alimentación se desarrolló aproximadamente en 1962;[5] la primera banda distribuidora fue una unidad de descarga lateral, usada en 1963 para colocar una capa de concreto en la vía rápida elevada de la 46a. Avenida Este en Denver, Colo. Poco después se desarrollaron los distribuidores radiales.

Las modificaciones y mejoras en los tres tipos básicos de transportadores han sido rápidas y significativas. A principios de los 50s las bandas transportadoras estaban limitadas a una capacidad que oscilaba entre 23 y 31 m³/hr. Las velocidades de colocación actuales –de 92 m³/hr en bandas de 45 cm de ancho y de 230 m³/hr en bandas de 61 cm– traen como resultado que las bandas transportadoras de concreto sean adecuadas lo mismo para efectuar colocaciones masivas de concreto que para construir edificios.

A medida que el diseño de bandas transportadoras fue evolucionando, se encontró que las características requeridas para evitar la segregación del concreto las hacían posible manejar concreto tanto de alto como de bajo revenimiento. Por regla, cualquier concreto con agregado normal o ligero, que pueda ser descargado por un camión-mezcladora, podrá colocarse por medio de una banda transportadora. También se han transportado con éxito concretos conteniendo agregado grueso de 80 y 150 mm en bandas de 41 y de 61 cm de ancho, respectivamente.[6]

CAPITULO 2

Consideraciones de diseño

2.1. Requerimientos generales

No todas las bandas transportadoras pueden colocar concreto con éxito. Deben ser diseñadas específicamente para superar los problemas que presenta el concreto.[7] Los transportadores de concreto operando a la velocidad de banda correcta y con las tolvas de carga, dispositivos de transferencia y limpiadores de banda funcionando adecuadamente, no modifican la resistencia, el revenimiento o el contenido de aire del concreto que transportan.[8,9] El diseño correcto de las bandas transportadoras de concreto destinadas para efectuar aplicaciones específicas depende del conocimiento de la interacción entre las diversas variables involucradas. La mayoría de los intentos fallidos en la colocación de concreto por medio de bandas transportadoras se puede atribuir a que en el diseño no se tomaron en cuenta los siguientes requerimientos de manejo y colocación:

1. Las dimensiones de todos los componentes del transportador deben estar de acuerdo al peso del concreto, especialmente la unidad impulsora, el bastidor de soporte y los rodillos guía de la banda. El concreto de peso normal es aproximadamente cincuenta por ciento más pesado que el de los materiales que se transportan usualmente, como los agregados.

2. El transportador en sí, o por lo menos el mecanismo de descarga del concreto, debe poder desplazarse por toda el área de colocación, sin interrumpir demasiado ni demorar la colocación del concreto. Esta condición es indispensable, ya que el concreto debe distribuirse uniformemente en toda el área de colocación. El desplazamiento debe ser mucho más rápido cuando se requiera que la colocación se efectúe en capas, para facilitar la compactación del concreto.

expuesto a las condiciones ambientales por muy poco tiempo. Durante la construcción de la Planta de Energía Castaic, en el sur de California, se transportó concreto a una distancia mucho mayor. En un día soleado, a 27°C de temperatura ambiente y sin protección sobre las bandas, la temperatura del concreto en estado plástico aumentó solamente 2°C, habiendo sido transportado 213 m.[21] El tiempo requerido para transportar el concreto a lo largo de todo el sistema de bandas fue de un minuto aproximadamente.

En la U. S. Army Engineer Waterways Experiment Station se llevaron a cabo experimentos en los que se simuló el transporte de concreto hasta una distancia de 1,800 m por medio de bandas transportadoras. Se estableció que no había cambios en la temperatura del concreto debidos a la transportación. La temperatura del concreto tendía a descender cuando era más elevada que la del aire; cuando era más baja que la del aire, entonces tendía a aumentar. La rapidez de ascenso o descenso dependía de la diferencia inicial de las temperaturas.[22]

Estos experimentos confirmaron algunos trabajos anteriores, que indicaban que el tiempo transcurrido (es decir, el tiempo tomado a partir de que el agua había sido agregada a la mezcla) tenía un efecto importante sobre el revenimiento del concreto.[23] Bajo condiciones de secado relativamente severas (temperaturas por encima de los 21°C), de humedad relativa menor al 50 por ciento y de velocidad del viento mayor a 16 km/hr, la pérdida en el revenimiento atribuible al transporte del concreto a una distancia de 450 m fue de aproximadamente 1.3 cm. El concreto transportado a más de 900 m experimentó una pérdida en el revenimiento más pronunciada, de aproximadamente 5 cm. Las pruebas efectuadas indicaron un aumento definitivo en la resistencia del concreto, correspondiente a la pérdida en el revenimiento. La disminución de aire incluido fue menor al 0.5 por ciento, en un concreto que originalmente contenía 5 por ciento de aire.

Cuando se vaya a usar un transportador de más de 450 m de longitud y se prevean condiciones ambientales extremas, será necesario contar con alguna forma de protección para mantener la trabajabilidad del concreto o para protegerlo contra la congelación. Las protecciones montadas sobre el transportador aumentan la carga muerta que debe ser soportada por su estructura y tal vez hagan necesarios algunos ajustes en el diseño estructural. Las protecciones autosoportantes aumentan la inversión de capital requerida para construir un sistema de transportación y generalmente sólo son prácticas cuando el transportador es fijo.

2.6. Características de la mezcla de concreto

Cualquier concreto estructural puede ser transportado satisfactoriamente por medio de una banda transportadora. Los concretos con revenimientos extremos, ya sean menores de 2.5 cm o mayores de 18 cm, tienden a reducir notablemente la capacidad de colocación de una banda transportadora. La tendencia a rodar del agregado de tamaño máximo mayor de 100 mm reduce mucho el ángulo de inclinación o de declinación permisible. Se puede obtener una colocación eficiente y una capacidad máxima de la banda transportadora si se utiliza una mezcla de concreto en estado plástico, homogénea y con un revenimiento controlado dentro de los 5 y 7.6 cm.[25] La velocidad de la banda se vuelve más crítica cuando el revenimiento no se encuentra dentro de este rango ideal. Generalmente, los concretos con revenimientos menores requieren bandas más lentas, mientras que los que poseen revenimientos altos necesitan bandas más rápidas.[26]

2.7. Especificaciones

Todos los factores tratados en detalle en los párrafos anteriores deben ser incorporados adecuadamente al diseño del transportador, con el fin de asegurar su funcionamiento satisfactorio. Ningún factor es tan importante como para producir por sí solo una operación satisfactoria o defectuosa. A pesar de que la segregación del concreto frecuentemente se atribuye a la velocidad de la banda transportadora, al diámetro de la polea y a la tensión de la banda, estos factores no siempre constituyen una causa principal para que se dé este problema. Se recomienda que las especificaciones de una banda transportadora de concreto vayan encaminadas a obtener los resultados deseados para el concreto ya colocado más que a delimitar los detalles específicos de su diseño. Las prácticas de campo recomendadas en el Capítulo 4 pueden incorporarse a las especificaciones del trabajo, en los casos que sean aplicables.

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CAPITULO 3.

Tipos de transportadores y sus funciones

3.1. Transportadores portátiles

La colocación del concreto a "corto alcance" o a "poca altura" hace necesario el uso de bandas transportadoras portátiles. Este equipo puede diferir de un fabricante a otro, pero todos tienen ciertas características básicas. La más importante es que cada unidad es autosuficiente y puede desplazarse fácilmente por toda la obra. Cada unidad debe tener su propia fuente de energía, ya que ningún equipo puede considerarse verdaderamente portátil si depende de una fuente de energía estacionaria. El peso y la movilidad necesarios en una banda transportadora portátil restringen su longitud total a 18 m aproximadamente. Esto, a su vez, limita la altura máxima de descarga a 11 m más o menos. La altura máxima de descarga está determinada por el ángulo máximo de inclinación al que puede manejarse eficientemente el concreto transportado en la banda (ver la Sección [illegible]).

Las bandas transportadoras portátiles son impulsadas por motores de gasolina utilizan sistemas de transmisión hidráulica para dar impulso a la banda. Estos sistemas de transmisión hidráulica poseen una alta potencia en relación al peso de la carga y tienen la capacidad de arrancar y parar con la banda cargada sin peligro de sufrir fallas mecánicas. Estos transportadores están equipados con un mecanismo de elevación por pluma y pueden ser autopropulsados y poseer dirección motriz.

Las bandas transportadoras portátiles (Fig. 2) colocan más concreto en un día que todos los demás tipos de transportadores juntos, debido a que la mayoría de los proyectos a base de concreto premezclado que requieren del manejo del material en la obra caen dentro del rango de "corto alcance" o "poca altura".

Una banda transportadora autopropulsada de 17 m de longitud total, con motor de 30 hp y dirección motriz, puede colocar el concreto a una velocidad mente cortas en el equipo de traslado, como son los camiones-mezcladoras y los grandes cucharones con descarga inferior usados en las plantas de presforzado y prefabricado. En la construcción de carreteras se usan bandas transportadoras de alta capacidad en combinación con el equipo de colocación de concreto para cimbra deslizante, para transferir el concreto desde los puntos accesibles a los camiones de volteo hasta el área de colocación. Los transportadores se usan también en la construcción de túneles para subir el concreto a las tolvas receptoras de las bombas. Aunque estas máquinas no caen completamente dentro de ninguno de los tres tipos de transportadores descritos anteriormente, operan de acuerdo a los límites y condiciones que se aplican por lo general a todas las bandas transportadoras de concreto.

3.6. Economías de la colocación por medio de transportadores

En el análisis final, la conveniencia de algún tipo de transportador para un proyecto en particular se determina en gran medida por lo que cuesta utilizar el equipo y por la inversión de capital que representa. El costo del uso de los transportadores generalmente se divide en dos clasificaciones: (1) operación o colocación y (2) instalación y mantenimiento.

Los fabricantes de sistemas de transportación de concreto afirman que las economías en sus costos de operación resultan de la colocación continua de concreto con tales equipos, en comparación con el método de la grúa y el cucharón con descarga inferior.[30]

Los costos de instalación de los transportadores tienden a resultar bastante independientes del volumen de concreto que será colocado. Están determinados en gran medida por el tipo de transportador utilizado y por la distancia a la que debe ser transportado el concreto. Por ejemplo, el uso de transportadores de alimentación para efectuar colocaciones de concreto de alcance y volumen significativos y sin restricciones en la rapidez de colocación, generalmente produce costos combinados más bajos que los que se pueden obtener con cualquier otro tipo de equipo de colocación.[31] Las losas de cimentación pesadas resultan excelentes para aplicarles concreto por medio de transportadores, pues requieren de la colocación de grandes volúmenes de concreto sin juntas frías.

El costo de mantenimiento de los transportadores es predominantemente el que resulta de mantener el equipo limpio y libre de acumulaciones de concreto. Es necesario reemplazar regularmente el material de la cuchilla raspadora de la banda sujeto a desgaste. El tiempo durante el cual el equipo ha estado en servicio es un factor determinante en la necesidad de reempalmar la banda y efectuar otros ajustes. El reemplazo de los deflectores y de los recubrimientos de hule en los puntos de transferencia varía, dependiendo del volumen y de la aspereza del concreto colocado.

En el proyecto de la Planta de Energía de Castaic, en California, se demostró otra ventaja de la colocación por medio de transportadores. Estos llevaban el concreto hasta lugares aparentemente inaccesibles, a más de 150 m³/hr y a un costo mucho menor al que se hubiera obtenido usando el sistema de grúa y cucharón con descarga inferior. La elevada rapidez de colocación permitió duplicar el tamaño de las colocaciones individuales, lo que originó una reducción en los costos de las cimbras.[32] *Cualquier* sistema de colocación de concreto que acelere la terminación total de una obra tendrá como resultado un ahorro considerable en los intereses de los préstamos para construcción. Por lo tanto, el propietario cuenta más rápidamente con los beneficios provenientes de la instalación completa.

Los transportadores fabricados expresamente para manejar concreto son relativamente baratos y pueden eliminar la necesidad de utilizar otros equipos más costosos, como son las grúas.[33] La inversión de capital requerida para instalar un transportador de concreto está determinada por la capacidad de colocación que se desea, así como por la distancia a la que debe operar. Un proyecto grande puede justificar la inversión en equipo necesaria para transportar el concreto por banda desde la planta de dosificación hasta todos los puntos del proyecto. En caso de que exista algún camino de acceso, los camiones-mezcladoras u otras unidades de traslado usualmente constituirán el método más económico para transportar el concreto desde la planta de dosificación hasta un punto razonablemente cercano al área de colocación. Debido a que normalmente resulta fácil incorporar soportes para los transportadores en combinación con las cimbras, o colocar soportes temporales en el área de colocación, la distancia que el transportador debe abarcar es generalmente menor de 30 m y los requerimientos de alcance en voladizo se limitan a menos de 12 m.

Los transportadores para colocar concreto se justifican frecuentemente aún en proyectos donde se requiere una grúa para otros fines que no sean la colocación de concreto. La colocación por medio de transportadores puede originar el uso de una grúa de menor capacidad. Además, el hecho de que la grúa esté disponible para llevar a cabo otros trabajos, en combinación con la capacidad de colocación de concreto de los transportadores, pueden permitir que el proyecto se termine con mayor rapidez.

CAPITULO 4

Prácticas de campo

4.1. Selección de los transportadores

Los principios de operación y diseño presentados anteriormente deben ser complementados con algunas reglas generales comprobadas en prácticas de campo, si se desea lograr un éxito total en la transportación del concreto. Debido a q los proyectos de construcción a base de concreto son relativamente cortos, comparados con la "vida" de una banda transportadora, generalmente no resulta práctico diseñar una banda transportadora para cada proyecto o aplicación especial. La práctica normal es seleccionar el equipo estándar comercialmente disponible, que posea una capacidad de colocación y de alcance adecuados, y organizar y planear su uso para satisfacer las secuencias constructivas generales requeridas para la realización correcta del trabajo. El diseño modular de la mayoría de los equipos hace posible alargar o acortar los trenes de alimentación, así como los transportadores de descarga lateral, de manera que las unidades individuales puedan adaptarse a las diferentes obras.

4.2 Capacidad real

La capacidad de colocación real de un transportador de concreto casi nunca será igual a la capacidad calculada por medio de la ecuación proporcionada en la Sección 2.5.5. Esto se debe principalmente a las inevitables demoras que ocurren durante la dosificación, el mezclado y el traslado del concreto a la banda transportadora. Otras demoras tienen lugar en la compactación y el acabado del concreto, así como durante el desplazamiento del transportador mismo. La magnitud de estas demoras variará de proyecto a proyecto y de día a día en la misma obra.

No es posible que una banda transportadora pueda colocar una cantidad de concreto mayor a su capacidad, pues el exceso de concreto transportado se derra-

CAPÍTULO 5

Métodos de control

5.1. Inspección del concreto

El hecho de que las bandas transportadoras de concreto constituyan un sistema abierto, en donde casi todo el concreto que está siendo colocado puede inspeccionarse visualmente, proporciona una excelente oportunidad para controlar el material. La franja de concreto que viaja sobre la banda debe inspeccionarse visualmente al principio y con frecuencia durante la colocación. El concreto es visible en cualquier punto de la banda y, debido a que ésta se mueve a una velocidad muy alta, el punto de inspección no es crítico.

Después de efectuar las pruebas adecuadas y someter el concreto a la inspección visual, para asegurar que a la banda solamente llega concreto de calidad, deberá ponerse especial atención en inspeccionar que la descarga del concreto desde los transportadores sea correcta y que su compactación también sea la adecuada. Las prácticas de campo recomendables deberán observarse cuidadosamente.

5.2. Pruebas

Si existe alguna duda con respecto a la capacidad de un determinado sistema para colocar concreto con éxito, las bandas transportadoras deben probarse bajo condiciones de trabajo antes de intentar cualquier colocación importante. Afortunadamente, el manejo de unos cuantos metros cúbicos de concreto en una banda transportadora confirmará el diseño adecuado e identificará las áreas problema. Cuando un sistema funciona satisfactoriamente en tales pruebas, no es arriesgado suponer que el transportador funcionará también en forma satisfactoria bajo condiciones reales de trabajo.

Las pruebas del concreto en estado plástico y las muestras para determinar su resistencia, tomadas durante las operaciones de descarga de los equipos mezcladores o de transporte y en el punto de descarga del transportador, deben asegurar su operación satisfactoria bajo cualquier condición de trabajo. La calidad del concreto que está siendo colocado en la estructura sólo puede medirse en el punto de colocación. Una vez establecida una correlación satisfactoria entre las muestras tomadas en el lugar de colocación y en el punto de descarga de las mezcladoras, el muestreo en el punto más favorable debe resultar satisfactorio, siempre que no se alteren las condiciones de colocación.

CAPITULO 6

Fuentes de información

6.1 Normas

Normas ACI

304-73 Recommended Practice for Measuring, Mixing, Transporting, Placing Concrete

Normas ASTM

C 94-74 Standard Specification for Ready-Mixed Concrete

C 1[illegible] 71 Standard Method of Sampling Fresh Concrete

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Guide to Formwork for Concrete

Reported by ACI Committee 347

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Objectives of safety, quality, and economy are given priority in these guidelines for formwork. A section on contract documents explains the kind and amount of specification guidance the engineer/architect should provide for the contractor. The remainder of the report advises the contractor on the best ways to meet the specification requirements safely and economically. Separate chapters deal with design, construction, and materials for formwork. Considerations peculiar to architectural concrete are also outlined in a separate chapter. Other sections are devoted to formwork for bridges, shells, mass concrete, and underground work. The concluding chapter on formwork for special methods of construction includes slipforming, preplaced aggregate concrete, tremie concrete, precast, and prestressed concrete.

Keywords: aggregates; aluminum; anchors (fasteners); architectural concrete; bridges (structures); canal linings; coatings; composite materials; concrete construction; construction costs; construction materials; culverts; falsework; fasteners; fiberboard; folded plates; form removal; formwork (construction); foundations; glass fibers; hangers; inserts; insulating boards; loads (forces); long span; lumber; mass concrete; multistory buildings; parting agents; plastic forms; plywood; precast concrete; preplaced aggregate concrete; pressure; prestressed concrete; quality control; reinforced concrete; roofs; safety; safety factor; settlement (structural); shells (structural forms); shelters; shoring; slipform construction; specifications; structural design; structural steels; subsurface structures; tolerances (mechanics); tunnels; underwater construction.

CONTENTS

This report replaces the standard ACI 347-78 (Reapproved 1984), which was withdrawn effective January 1989.

*Chairman of the committee when much of the work on this report was done. Daniel Litt, associate member of the committee, contributed significantly to the section on slipforming.

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tion of conduits and pipes embedded in concrete according to ACI 318 (Section 6.3).

u. Temporary openings or attachments for climbing crane or other material handling equipment.

2.2—Loads

2.2.1 *Vertical loads*—Vertical loads consist of dead load and live load. The weight of formwork plus the weight of freshly placed concrete is dead load. The live load includes the weight of workmen, equipment, material storage, runways, and impact.

Vertical loads assumed for shoring and reshoring design for multistory construction must include all loads transmitted from the floors above as dictated by the proposed construction schedule. Refer to Section 2.5, Shores.

Vertical supports and horizontal framing should be designed for a minimum live load of 50 psf of horizontal projection. When motorized carts are used the minimum live load should be 75 psf.

The minimum design load for combined dead and live loads should be 100 psf, or 125 psf if motorized carts are used.

2.2.2 *Lateral pressure of concrete*—Unless the conditions of Section 2.2.2.1 or 2.2.2.2 are met, formwork should be designed for the lateral pressure of the newly placed concrete given in Eq. (2-1). Maximum and minimum values given for other pressure formulas do not apply to Eq. (2-1) (see Appendix for metric conversions of equations in this section).

$$p = wh \tag{2-1}$$

where p = lateral pressure, psf; w = unit weight of fresh concrete, pcf; and h = depth of fluid or plastic concrete, ft.

For columns or other forms that may be filled rapidly before any stiffening of the concrete takes place, h should be taken as the full height of the form, or the distance between construction joints when more than one placement of concrete is to be made.

2.2.2.1 For concrete made with Type I cement,* weighing 150 pcf, containing no pozzolans or admixtures, having a slump of 4 in. or less and normal internal vibration to a depth of 4 ft or less, formwork may be designed for a lateral pressure as follows, where R = rate of placement, ft per hr; and T = temperature of concrete in the form, deg F.

FOR COLUMNS

$$p = 150 + 9000\,R/T \tag{2-2}$$

with a maximum of 3000 psf, a minimum of 600 psf, but in no case greater than 150 h.

FOR WALLS with rate of placement less than 7 ft per hr

$$p = 150 + 9000\,R/T \tag{2-2a}$$

with a maximum of 2000 psf, a minimum of 600 psf, but in no case greater than 150 h.

FOR WALLS with a rate of placement of 7 to 10 ft per hr

$$p = 150 + 43{,}400/T + 2800\,R/T \tag{2-3}$$

with a maximum of 2000 psf, a minimum of 600 psf, but in no case greater than 150 h.

2.2.2.2. Alternatively, a method based on appropriate experimental data may be used to determine the lateral pressure used for form design (References 2.1 through 2.5).

2.2.2.3 If concrete is pumped from the base of the form, the form should be designed for full hydrostatic head of concrete wh plus a minimum allowance of 25 percent for pump surge pressure. In certain instances pressures may be as high as the face pressure of the pump piston.

2.2.2.4 Caution must be taken when using external vibration or concrete made with shrinkage compensating or expansive cements. Pressures in excess of equivalent hydrostatic may occur.

2.2.2.5 For slipform lateral pressures, see Section 7.3.2.4.

2.2.3 *Horizontal loads*—Braces and shores should be designed to resist all foreseeable horizontal loads such as seismic forces, wind, cable tensions, inclined supports, dumping of concrete, and starting and stopping of equipment. Wind loads on enclosures or other wind breaks attached to the formwork should be considered in addition to these loads.

2.2.3.1 For building construction, in no case should the assumed value of horizontal load due to wind, dumping of concrete, inclined placement of concrete, and equipment acting in any direction at each floor line be less than 100 lb per linear ft of floor edge or 2 percent of total dead load on the form distributed as a uniform load per linear foot of slab edge, whichever is greater.

2.2.3.2 Wall form bracing should be designed to meet the minimum wind load requirements of ANSI A58.1 or of the local building code, whichever is more stringent. For wall forms exposed to the elements, the minimum wind design load should not be less than 15 psf. Bracing for wall forms should be designed for a horizontal load of at least 100 lb per linear ft of wall, applied at the top.

2.2.3.3 Wall forms of unusual height or exposure should be given special consideration.

2.2.4 *Special loads*—The formwork should be designed for any special conditions of construction likely to occur, such as unsymmetrical placement of concrete, impact of machine-delivered concrete, uplift, concentrated loads of reinforcement, form handling loads, and storage of construction materials. Form designers should be alert to provide for special loading condi-

*The committee has insufficient test data with other cem[illegible]pes. See original statement of formulas in Reference 1.2.

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tions, such as walls constructed over spans of slabs or beams which exert a different loading pattern before hardening of concrete than that for which the supporting structure is designed.

Imposition of any construction loads on the partially completed structure should not be allowed except as specified in formwork drawings or with the approval of the engineer or architect. See Section 3.8 for special conditions pertaining to multistory work.

2.2.5 *Post-tensioning loads*—Shores, reshores, and backshores need to be analyzed for both concrete placement loads and for all load transfer that takes place during post-tensioning.

2.3—Unit stresses

Unit stresses for use in the design of formwork, exclusive of accessories, are given in the applicable codes or specifications listed in Chapter 4. When fabricated formwork, shoring, or scaffolding units are used, manufacturer's recommendations for allowable loads may be followed if supported by the test reports of a qualified and recognized testing agency or successful experience records; for formwork materials which will experience substantial reuse, reduced values should be used. For formwork materials with limited reuse, allowable stresses specified in the appropriate design codes or specifications for temporary structures or for temporary loads on permanent structures may be used. Where there will be a considerable number of formwork reuses or where formwork is fabricated from materials such as steel, aluminum, or magnesium, it is recommended that the formwork be designed as a permanent structure carrying permanent loads.

2.4—Safety factors for accessories

Table 2.4 shows recommended minimum factors of safety for formwork accessories such as form ties, form anchors, and form hangers. In selecting these accessories, the formwork designer should be certain that materials, furnished for the job meet these minimum ultimate strength safety requirements.

Table 2.4 — Minimum safety factors of formwork accessories*

Accessory	Safety factor	Type of construction
Form tie	2.0	All applications
Form anchor	2.0	Formwork supporting form weight and concrete pressures only
	3.0	Formwork supporting weight of forms, concrete, construction live loads, and impact
Form hangers	2.0	All applications
Anchoring inserts used as form ties	2.0	Precast concrete panels when used as formwork

*Safety factors are based on ultimate strength of accessory.

2.5—Shores

2.5.1 *General*—Shores are defined as vertical or inclined support members designed to carry the weight of formwork, concrete, and construction loads above.

2.5.2 *Splices*—Field-constructed butt or lap splices of timber shoring are not recommended unless they are made using fabricated hardware devices of demonstrated strength and stability. If plywood or lumber splices are made for timber shoring, they should be designed against buckling and bending as for any other structural compression member.

2.5.3 *Multistory structures*—Prior to construction, an overall plan for scheduling of shoring and reshoring or backshoring, and calculation of loads transferred to the structure, should be prepared by a qualified and experienced formwork designer. The structure's capacity to carry these loads should be reviewed or approved by the engineer/architect. The plan and responsibility for its execution remain with the contractor.

Shores and reshores or backshores (as defined in Section 3.8) must be designed to carry all loads transmitted to them. A rational analysis should be used to determine the number of floors to be shored, reshored, or backshored and to determine the loads transmitted to the floors, shores, and reshores or backshores as a result of the construction sequence.

The analysis should consider, but should not necessarily be limited to, the following:

1. Structural design load of the slab or member including live load, partition loads, and other loads for which the engineer designed the slab. Where the engineer included a reduced live load for the design of certain members and allowances for construction loads, such values should be shown on the structural drawings and be taken into consideration when performing this analysis.
2. Dead load weight of the concrete and formwork.
3. Construction live loads, such as placing crews and equipment or stored materials.
4. Design strength of concrete specified.
5. Cycle time between placement of successive floors.
6. Strength of concrete at time it is required to support shoring loads from above.
7. The distribution of loads between floors, shores, and reshores or backshores at the time of placing concrete, stripping formwork, and removal of reshoring or backshoring.[2.6,2.7]
8. Span of slab or structural member between permanent supports.
9. Type of formwork systems, i.e., span of horizontal formwork components, individual shore loads, etc.
10. Minimum age where appropriate.

Commercially available test cells can be placed under selected shores to monitor actual shore loads to guide the shoring and reshoring process as construction proceeds.[2.8]

2.6—Bracing and lacing

The formwork system should be designed to transfer all horizontal loads to the ground or to completed construction in such a manner as to insure safety at all times. Diagonal bracing should be provided in vertical and horizontal planes where required to resist lateral loads and to prevent instability of individual members.

Horizontal lacing may be considered in design to hold in place and increase the buckling strength of individual shores and reshores or backshores. Lacing should be provided in whatever directions are necessary to produce the correct slenderness ratio l/r for the load supported, where l = unsupported length and r = least radius of gyration. The braced system should be anchored in a manner to insure stability of the total system.

2.7—Foundations for formwork

Proper foundations on ground such as mudsills, spread footings, or pile footings should be provided. If soil under mudsills is or may become incapable of supporting superimposed loads without appreciable settlement, it should be stabilized or other means of support should be provided. No concrete should be placed on formwork supported on frozen ground.

2.8—Settlement

Formwork should be so designed and constructed that vertical adjustments can be made to compensate for take-up and settlements.

2.9—References

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2.6. Grundy, Paul, and Kabaila, A., "Construction Loads on Slabs with Shored Formwork in Multistory Buildings," ACI JOURNAL, *Proceedings* V. 60, No. 12, Dec. 1963, pp. 1729-1738.

2.7. Agarwal, R. K., and Gardner, Noel J., "Form and Shore Requirements for Multistory Flat Slab Type Buildings," ACI JOURNAL, *Proceedings* V. 71, No. 11, Nov. 1974, pp. 559-569.

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CHAPTER 3—CONSTRUCTION

3.1—Safety precautions

Constructors should follow all state, local, and federal codes, ordinances, and regulations pertaining to forming and shoring.

In addition to the very real moral and legal responsibility to maintain safe conditions for workmen and the public, safe construction is in the final analysis more economical than any short-term cost savings from cutting corners on safety provisions. Attention to safety is particularly significant in formwork construction as these structures support the concrete during its plastic state and as it is developing strength, until the concrete becomes structurally self-sufficient. Following the design criteria contained in this guide is essential to assuring safe performance of the forms. All structural members and connections should be carefully planned so that a sound determination of loads may be accurately made and stresses calculated.

In addition to the adequacy of the formwork, special structures such as multistory buildings require consideration of the behavior of newly completed beams and slabs which are used to support formwork and other construction loads. It must be kept in mind that the strength of freshly cast slabs or beams is less than that of an aged slab.

Formwork failures can be attributed to human error, substandard materials and equipment, omission and basic inadequacy in design. Careful supervision and continuous inspection of formwork erection can prevent many accidents.

Construction procedures must be planned in advance to insure the safety of personnel and the integrity of the finished structure. Some of the safety provisions which should be considered are:

a. Erection of safety signs and barricades to keep unauthorized personnel clear of areas in which erection, concrete placing, or stripping is under way.

b. Providing experienced form watchers during concrete placement to assure early recognition of possible form displacement or failure. A supply of extra shores or other material and equipment that might be needed in an emergency should be readily available.

c. Provision for adequate illumination of the formwork.

d. Inclusion of lifting points in the design and detailing of all forms which will be crane-handled. This is especially important in flying forms or climbing forms. In the case of wall formwork, consideration should be given to an independent scaffold bolted to the previous lift.

e. Incorporation of scaffolds, working platforms, and guardrails into formwork design and all formwork drawings.

f. A program of field safety inspections of formwork.

3.1.1 Some common construction deficiencies which may lead to form failures are these, which are applicable to all formwork:

a. Failure to inspect formwork during and after concrete placement to detect abnormal deflections or other signs of imminent failure which could be corrected.

b. Insufficient nailing, bolting, or fastening.

c. Insufficient or improper lateral bracing.

d. Failure to comply with manufacturer's recommendations.

e. Failure to construct formwork in accordance with the form drawings.

f. Lack of proper field inspection by qualified persons to assure that form design has been properly interpreted by form builders.

g. Use of lumber containing knots that impair the strength of the member.

h. Improper welding of structural components.

3.1.2 Construction deficiencies applicable to vertical formwork include:

a. Failure to control rate of placing concrete vertically without regard to design parameters.

b. Inadequately tightened or secured form ties or hardware.

c. Form damage in excavation by reason of embankment failure.

d. Use of external vibrators on forms not designed for their use.

e. Deep vibrator penetration of earlier semi-hardened lifts.

f. Improper framing of blockouts.

g. Improperly constructed or located pouring pockets.

h. Inadequate bulkheads.

i. Improperly anchored top forms on a sloping face.

j. Failure to provide adequate support for lateral pressures on formwork.

k. Attempt to plumb forms against concrete pressure force.

3.1.3 Construction deficiencies applicable to horizontal forms for suspended structures include:

a. Improper use of multi-tier shores.

b. Failure to regulate properly the rate and sequence of placing concrete horizontally to avoid unanticipated loadings on the formwork.

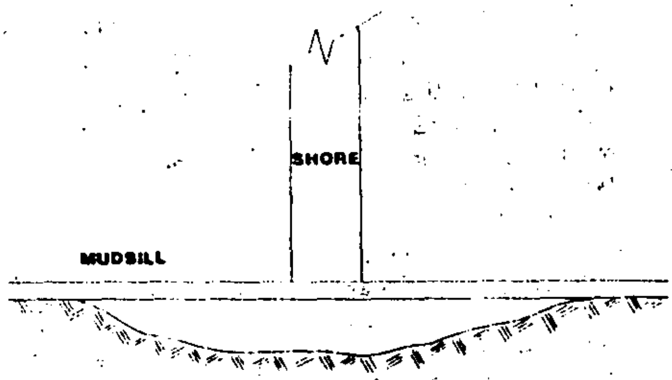

Fig. 3.1.3.a—Inadequate bearing under mudsill

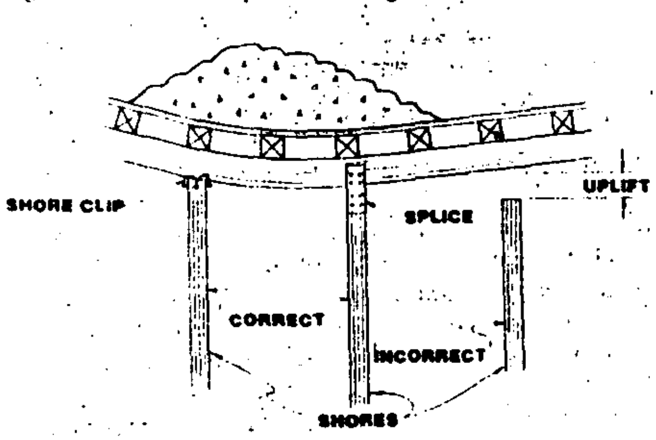

Fig. 3.1.3.b—Uplift of formwork. Connection of shores to joists and stringers must hold shores in place when uplift or torsion occurs. Lacing to reduce the shore or slenderness ratio may be required in both directions

c. Shoring not plumb, thus inducing lateral loading as well as reducing vertical load capacity.

d. Locking devices on metal shoring not locked, inoperative, or missing.

e. Vibration from adjacent moving loads or load carriers.

f. Inadequately tightened or secured shore hardware or wedges.

g. Loosening of reshores or backshores under floors below.

h. Premature removal of supports, especially under cantilevered sections.

i. Inadequate bearing or unsuitable soil under mudsills (Fig. 3.1.3.a).

j. Mudsills placed on frozen ground subject to thawing.

k. Connection of shores to joists, stringers, or wales which are inadequate to resist uplift or torsion at joints (see Fig. 3.1.3.b).

l. Failure to consider effects of load transfer which may occur during post-tensioning (see Section 3.8.7).

m. Inadequate shoring and bracing of composite construction.

3.2—Construction practices and workmanship

3.2.1 *Fabrication and assembly details*

3.2.1.1 Studs, wales or shores should be properly spliced.

3.2.1.2 Joints or splices in sheathing, plywood panels, and bracing should be staggered.

3.2.1.3 Shores should be installed plumb and with adequate bearing.

3.2.1.4 Use specified size and capacity of form ties or clamps.

3.2.1.5 Install and properly tighten all form ties or clamps as specified. All threads should fully engage the nut or coupling.

3.2.1.6 Forms should be sufficiently tight to prevent loss of mortar from the concrete.

3.2.1.7 Access holes may be necessary in wall forms or other high, narrow forms to facilitate concrete placement.

3.2.2 *Joints in the concrete*

3.2.2.1 Contraction joints, construction joints, and isolation joints should be installed as specified (see Fig. 3.2.2.1).

3.2.2.2 Bulkheads for control joints or construction joints should preferably be made by splitting along the lines of reinforcement passing through the bulkhead so that each portion may be positioned and removed separately without applying undue pressure on the reinforcing rods, which could cause spalling or cracking of the concrete. When required on the engineer/architect's drawing, beveled inserts at control joints must be left undisturbed when forms are stripped, and removed only after the concrete has been sufficiently cured. Wood strips inserted for architectural treatment should be kerfed to permit swelling without causing pressure on the concrete.

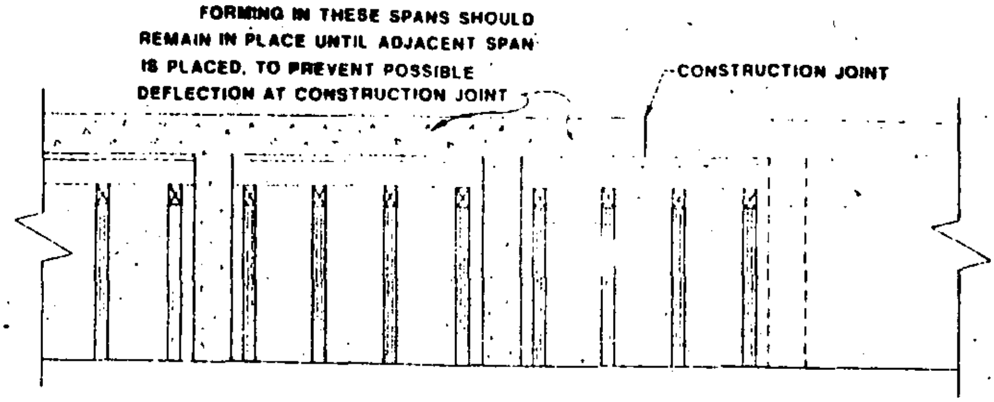

Fig. 3.2.2.1—Forming and shoring restraints at construction joints in supported slabs

3.2.3 *Sloping surfaces*—Sloped surfaces steeper than 1.5 horizontal to 1 vertical should be provided with a top form to hold the shape of the concrete during placement, unless it can be demonstrated that the top forms can be omitted.

3.2.4 *Inspection*

3.2.4.1 Forms should be inspected and checked before the reinforcing steel is placed to insure that the dimensions and the location of the concrete members will conform to the drawings.

3.2.4.2 Blockouts, inserts, sleeves, anchors, and other embedded items should be properly identified, positioned, and secured.

3.2.4.3 Forms should be checked for camber when specified.

3.2.5 *Cleanup and coatings*

3.2.5.1 Forms should be thoroughly cleaned of all dirt, mortar, and foreign matter and coated with a release agent before each use. Where the bottom of the form is inaccessible from within, access panels should be provided to permit thorough removal of extraneous material before placing concrete. If surface appearance is important, forms should not be reused after damage from previous use has reached the state of possible impairment to concrete surfaces.

3.2.5.2 Form coatings should be applied before placing of reinforcing steel and should not be used in such quantities as to run onto bars or concrete construction joints.

3.2.6 *Construction operations on the formwork*

3.2.6.1 Building materials including concrete must not be dropped or piled on the formwork in such manner as to damage or overload it.

3.2.6.2 Runways for moving equipment should be provided with struts or legs as required and should be supported directly on the formwork or structural member. They should not bear on nor be supported by the reinforcing steel unless special bar supports are provided. The formwork must be suitable for the support of such runways without significant deflections, vibrations, or lateral movements.

3.2.7 *Loading new slabs*—Guard against overloading of new slabs. Loads such as aggregate, timber, boards, reinforcing steel, or support devices must not be placed on new construction in such manner as to damage or overload it.

3.3—Tolerances

Tolerance is a permissible variation from lines, grades, or dimensions given in contract drawings. Suggested tolerances for concrete structures can be found in ACI 117.

The contractor is expected to set and maintain concrete forms so as to insure completed work within the tolerance limits.

3.3.1 *Recommendations for engineer/architect and contractor*—Tolerances should be specified by the engineer/architect so that the contractor will know precisely what is required and can design and maintain his formwork accordingly. It should be remembered that specifying tolerances more exacting than needed may increase construction costs.

Contractors are expected, and should be required, to establish and maintain in an undisturbed condition until final completion and acceptance of a project, control points and bench marks adequate for their own use and for reference to establish tolerances. (This requirement may become even more important for the contractor's protection when tolerances are not specified or shown). The engineer/architect should specify tolerances or require performance within generally accepted limits. Where a project involves features sensitive to the cumulative effect of generally accepted tolerances on individual portions, the engineer/architect should anticipate and provide for this effect by setting a cumulative tolerance. Where a particular situation involves several types of generally accepted tolerances on items such as concrete, location of reinforcement, and fabrication of reinforcement, which become mutually incompatible, the engineer/architect should anticipate the difficulty and specify special tolerances or indicate which controls. The contract specifications should clearly state that a permitted variation in one part of the construction or in one section of the specifications must not be construed as permitting violation of the more stringent requirements for any other part of the construction or in any other such specification section.

The engineer/architect should be responsible for coordinating the tolerances for concrete work with the requirements of other trades whose work adjoins the concrete construction.

3.4—Irregularities in formed surfaces

This section provides a way of evaluating surface variations due to forming quality, but is not intended to apply to surface defects such as bugholes (blowholes) and honeycomb attributable to placing and consolidation deficiencies. The latter are more fully explained by ACI 309.2-R. Allowable irregularities are designated either abrupt or gradual. Offsets and fins resulting from displaced, mismatched, or misplaced forms, sheathing, or liners or from defects in forming materials are considered abrupt irregularities. Irregularities resulting from warping and similar uniform variations from planeness or true curvature are considered gradual irregularities.

Gradual irregularities should be checked with a 5-ft template, consisting of a straightedge for plane surfaces or a shaped template for curved or warped surfaces. In measuring irregularities, the straightedge or template may be placed anywhere on the surface in any direction.

Four classes of formed surface are defined in Table 3.4. The engineer/architect should indicate which is required for the work he is specifying.

Table 3.4 — Permitted irregularities in formed surfaces checked with a 5-ft template

Type of irregularity	Class of surface			
	A	B	C	D
Gradual	⅛ in.	¼ in.	½ in.	in.
Abrupt	⅛ in.	¼ in.	¼ in.	1 in.

Class A is suggested for surfaces prominently exposed to public view, where appearance is of special importance. Class B is intended for coarse-textured concrete formed surfaces intended to receive plaster, stucco, or wainscoting. Class C is a general standard for permanently exposed surfaces where other finishes are not specified. Class D is a minimum quality requirement for surfaces where roughness is not objectionable, usually applied where surfaces will be permanently concealed. Special limits on irregularities may be needed for surfaces continuously exposed to flowing water, drainage, or exposure. If permitted irregularities are different from those given in Table 3.4, they should be specified by the engineer/architect.

3.5—Shoring and centering

3.5.1 *Shoring*—Shoring must be supported on satisfactory foundations such as spread footings, mudsills, or piling as discussed in Section 2.7.

Shoring resting on intermediate slabs or other construction already in place need not be located directly above shores or reshores below unless the slab thickness and the location of its reinforcement are inadequate to take the reversal of stresses and punching shear. Where the latter conditions are questionable, the shoring location should be approved by the engineer/architect (see Fig. 3.5.1.a).

All members must be straight and true without twists or bends. Special attention should be given to beam and slab, or one-way and two-way joist construction to prevent local overloading when a heavily loaded shore rests on the thin slab.

Multitier shoring is not recommended and is considered a dangerous practice.

Where a slab load is supported on one side of the beam only, edge beam forms should be ca-

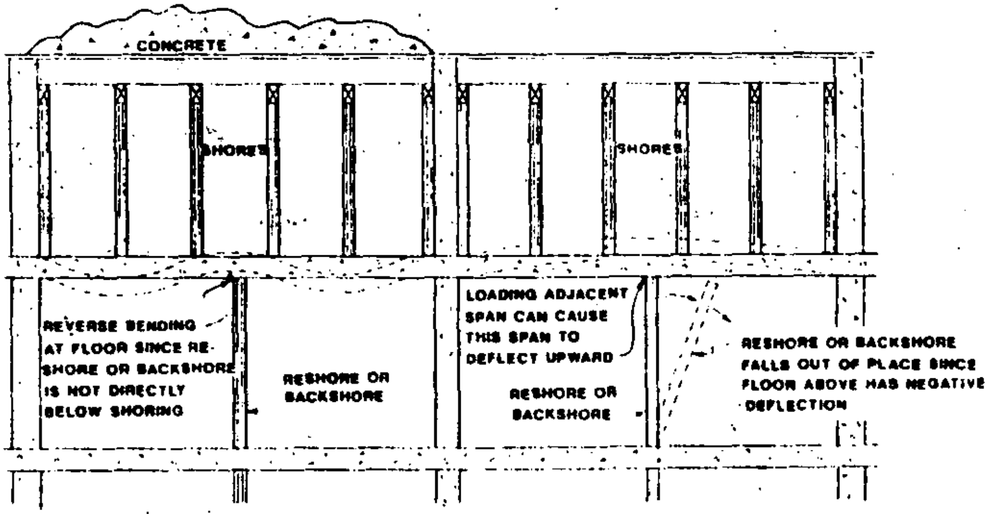

Fig. 3.5.1.a—Reshore installation. Improper positioning of shores from floor to floor may create bending stresses for which the slab was not designed. If reshores or backshores do not match the shores above, then calculate for reversal stresses. Generally, the dead load stresses are sufficient to compensate for reversal stresses caused by reshores. Reshores and backshores must be prevented from falling.

45

planned to prevent tipping of the beam due to unequal loading.

Vertical shores must be erected so that they cannot tilt, and must have firm bearing. Inclined shores must be braced securely against slipping or sliding. The bearing ends of shores should be square. Connections of shore heads to other framing should be adequate to prevent the shores from falling out when reversed bending causes upward deflection of the forms (see Fig. 3.1.3.b).

3.5.2 *Centering*—When centering is used, lowering is generally accomplished by the use of sand boxes, jacks, or wedges beneath the supporting members. For the special problems associated with the construction of centering for folded plates, thin shells, and long span roof structures, see Section 6.4.

3.5.3 *Shoring for composite action between previously erected steel or concrete framing and cast-in-place concrete*—See Section 6.3.

3.6—Inspection and adjustment of formwork*

3.6.1 *Before concreting*

3.6.1.1 Telltale devices should be installed on shores or forms to detect formwork movements during concreting.

3.6.1.2 Wedges used for final alignment before concrete placement should be secured in position before the final check.

*Helpful information about forms before, during, and after concreting may be found in Reference 1.3 and the ACI *Manual of Concrete Inspection*, SP-2.

3.6.1.3 Formwork must be anchored to the shores below so that movement of any part of the formwork system will be prevented during concreting.

3.6.1.4 Additional elevation of formwork should be provided to allow for closure of form joints, settlements of mudsills, shrinkage of lumber, and elastic shortening and dead load deflections of form members

3.6.1.5 Positive means of adjustment (wedges or jacks) should be provided to permit realignment or readjustment of shores if settlement occurs.

3.6.2 *During and after concreting*—During and after concreting, but before initial set of the concrete, the elevations, camber, and plumbness of formwork systems should be checked, using telltale devices.

Formwork must be continuously watched so that any corrective measures found necessary may be promptly taken. Form watchers must always work under safe conditions and should establish in advance a method of communication with placing crews in case of emergency.

3.7—Removal of forms and supports

3.7.1 *Discussion*—Although the contractor is generally responsible for design, construction, and safety of formwork, it is recommended that criteria for removal of forms or shores be specified by the engineer/architect.

3.7.2 *Recommendations*

3.7.2.1 The engineer/architect should specify the minimum strength of the concrete to be attained before

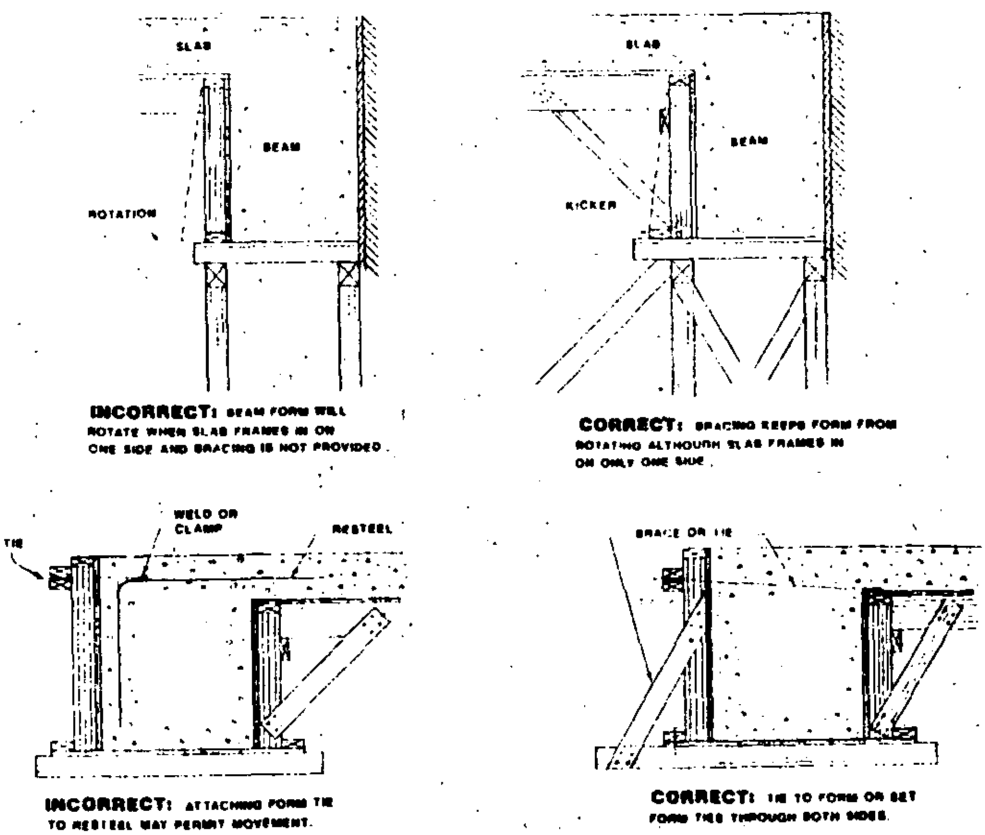

Fig. 3.5.1.b—Prevention of rotation is important where the slab frames into the beam form on only one side

removal of forms or shores. The strength may be determined by tests on job-cured specimens or on the in-place concrete. The engineer/architect should specify who will make the specimens and who will make the tests.

Results of such tests, as well as records of weather conditions and other pertinent information, should be recorded. Depending on the circumstances, a minimum elapsed time after concrete placement may be established for removal of the formwork.

Determination of the time of form removal should be based on the resulting effect on the concrete.* When forms are stripped there must be no excessive deflection or distortion and no evidence of damage to the concrete, due either to removal of support or to the stripping operation (Fig. 3.7.2.1). When forms are removed before the specified curing is completed, measures should be taken to continue the curing and provide adequate thermal protection for the concrete. Supporting forms and shores must not be removed from beams, floors, and walls until these structural units are strong enough to carry their own weight and any approved superimposed load. In no case should supporting forms and shores be removed from horizontal members before concrete strength is at least 70 percent of design strength, as determined by field-cured cylinders or other approved methods, unless approved by the engineer/architect.

As a general rule, the forms for columns and piers may be removed before those for beams and slabs.

*Helpful information on strength development of concrete under varying conditions of temperature and with various admixtures may be found in ACI 305R and ACI 306R.

Formwork and shoring should be constructed so each can be easily and safely removed without impact or shock to permit the concrete to carry its share of the load gradually and uniformly.

3.7.2.2 When field operations are controlled by the engineer/architect's specifications, the removal of forms, supports, and protective enclosures, and the discontinuance of heating and curing must follow the requirements of the contract documents. When standard beam or cylinder tests are used to determine stripping times, test specimens should be cured under conditions which are not more favorable than the most unfavorable conditions for the portions of the concrete which the test specimens represent. The curing records may serve as the basis on which the engineer/architect will determine his approval of form stripping.

3.7.2.3 Since the minimum stripping time is a function of concrete strength, the preferred method of determining stripping time is by the use of tests of job-cured specimens or of the concrete in place. However, when the engineer/architect does not specify minimum strength required of concrete at the time of stripping, the following elapsed times may be used under ordinary conditions. The times shown represent cumulative number of days, or hours, not necessarily consecutive, during which the temperature of the air surrounding the concrete is above 50 F. If high-early-strength concrete is used, these periods may be reduced as approved by the engineer/architect. Conversely, if ambient temperatures remain below 50 F, or if retarding agents are used, then these periods should be increased at the discretion of the engineer/architect.

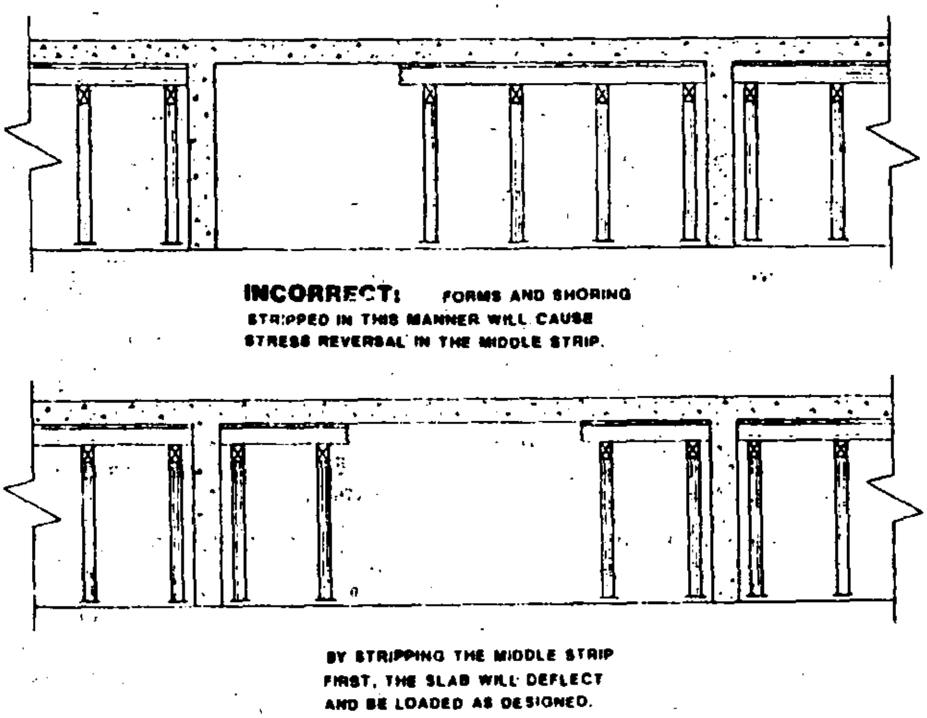

Fig. 3.7.2.1—Stripping sequence for two-way slabs

Walls*	12 hr	
Columns*	12 hr	
Sides of beams and girders*	12 hr	
Pan joist forms[†]		
30 in. wide or less	3 days	
Over 30 in. wide	4 days	

	Structural live load less than structural dead load	Structural live load more than structural dead load
Arch centers	14 days	7 days
Joist, beam, or girder soffits		
Under 10 ft clear span between structural supports	7 days[‡]	4 days
10 to 20 ft clear span between structural supports	14 days[‡]	7 days
Over 20 ft clear span between structural supports	21 days[‡]	14 days
One-way floor slabs		
Under 10 ft clear span between structural supports	4 days[‡]	3 days
10 to 20 ft clear span between structural suports	7 days[‡]	4 days
Over 20 ft clear span between structural supports	10 days[‡]	7days

Two-way slab systems[§]. . . Removal times are contingent on reshores where required, being placed as soon as practicable after stripping operations are complete but not later than the end of the working day in which stripping occurs. Where reshores are required to implement early stripping while minimizing sag or creep (rather than for distribution of superimposed construction loads as covered in Section 3.8), capacity and spacing of such reshores should be specified by the engineer/architect.

Post-tensioned slab system[§]. . . As soon as full post-tensioning has been applied.

3.8—Shoring and reshoring of multistory structures

3.8.1 *Discussion*—Multistory work represents special conditions, particularly in relation to removal of forms and shores. Reuse of form material and shores is an obvious economy. Furthermore, the speed of construction customary in this type of work provides the additional advantage of permitting other trades to follow concreting operations from floor to floor as closely as possible. However, the shoring which supports green concrete is necessarily supported by lower floors which may not be designed for these loads. For this reason shoring must be provided for a sufficient number of floors to develop the necessary capacity to support the imposed loads without excessive stress or deflection.

Reshoring and *backshoring* are procedures used to distribute construction loads through the lower floors. Though load distribution analysis is similar for the two, there are significant differences in magnitude, duration, and timing of floor and shore loads for the two procedures.

For purposes of this discussion the following definitions apply:

Shores—Vertical or inclined support members designed to carry the weight of formwork, concrete, and construction loads above.

Reshores—Shores placed snugly under a stripped concrete slab or structural member after the original forms and shores have been removed from a large area, thus requiring the new slab or structural member to deflect and support its own weight and existing construction loads applied prior to the installation of the reshores. It is assumed that the reshores carry no load at the time of installation. Afterward, additional construction loads will be distributed among all members connected by reshores.

Backshores—Shores placed snugly under a stripped concrete slab or structural member after the original formwork and shores have been removed from a small area without allowing the slab to deflect or support its own weight or existing construction loads from above. It is assumed that the backshores carry the same load as that carried by the original shores they replace. Original shores left in place with or without removal of the forms act in the same way as backshores. Added construction loads will be distributed among all members interconnected by backshores after the first level support at grade is removed.

Table 3.8.1 compares key features of reshoring and backshoring. With *backshoring*, so long as the first level shores remain in place in contact with grade, each tier of shores must carry the weight of all concrete and construction loads above it. This may be the weight of several floors. While *reshoring* remains in place at grade level, accumulated shore loads are less because each slab has been permitted to carry its own weight before reshores were put in place.

Once the tier of backshores or reshores in contact with grade has been removed, the assumption is made that the system of slabs behaves elastically. The slabs interconnected by reshores or backshores will deflect

Table 3.8.1 — Comparison of reshoring and backshoring

BACKSHORING	RESHORING
Strip small areas.	Strip several entire bays.
Do not let slab deflect.	Allow slab to deflect.
Install backshores before any further stripping occurs.	Install reshores without removing deflection.
Slab does not carry its own weight.	Slabs carry their own weight.
Backshores have an initial load.	Reshores have no initial load.

*Where such forms also support formwork for slab or beam soffits, the removal times of the latter should govern.

†Of the type which can be removed without disturbing forming or shoring.

‡Where forms may be removed without disturbing shores, use half of values shown but not less than 3 days.

§See Section 3.8 for special conditions affecting number of floors to remain shored or reshored.

equally during addition or removal of loads. Loads will be distributed among the slabs in proportion to their developed stiffness. Addition or removal of loads may be due to construction activity or to removing shores, reshores, or backshores in the system.

3.8.2 *Advantages of the two systems*

Reshores—Stripping formwork is more economically accomplished if all the material can be removed at the same time and moved from the area before placing reshores. Slabs are allowed to support their own weight, thus reducing the load in the reshores. Reshoring usually requires fewer levels of interconnected slabs, thus freeing more areas for other trades. Near-capacity loads in slabs usually occur for shorter periods.

Backshores—Stripping of forms may be accomplished at an earlier age because large areas of concrete are not required to carry their own weight. New slabs carry less load, thus reducing the effects of early creep. Using the original shores in place of backshores avoids the special attention required to assure that backshores are placed uniformly tight under the slab. It also provides better assurance that shores are placed in the same pattern on each floor.

3.8.3 *Design*—Refer to Chapter 2.

3.8.4 *Placing reshores or backshores*—When used in this section, the word *shore* refers to either backshores, reshores, or original shores.

Reshoring or backshoring is one of the most critical operations in formwork; consequently the procedure should be planned in advance and should be reviewed or approved by the engineer/architect. Operations should be performed so that at no time will areas of new construction be required to support combined dead and construction loads in excess of their capability as determined by design load and developed concrete strength at the time of stripping and reshoring or backshoring.

In no case should shores be so located as to significantly alter the pattern of stress determined in the structural analysis or to induce tensile stresses where reinforcing bars are not provided. Size and number of shores, and bracing if required, must provide a supporting system capable of carrying any loads that may possibly be imposed on it.

Where possible, shores should be located in the same position on each floor so that they will be continuous in their support from floor to floor. When shores above are not directly over shores below, an analysis should be made to determine whether or not detrimental stresses are produced in the slab. This condition seldom occurs in reshoring because the bending stresses normally caused by the offset reshores are not large enough to overcome the stress pattern that has already been established as a result of the slab carrying its own dead load. When backshoring is used there is no initial stress pattern established. Therefore the stress pattern set up by the offset backshores becomes primary and may produce tension stresses in areas without reinforcing bars. Where slabs are designed for light live loads, or on long spans where the loads on the shores are heavy, care should be used in placing these shores so that the loads on the shores do not cause excessive punching shear or bending stress in the slab.

While reshoring is under way, no construction loads should be permitted on the new construction unless the new construction can safely support the construction loads.

When placing reshores, care should be taken not to preload the lower floor and also not to remove the normal deflection of the slab above. The reshore is simply a strut and should be tightened only to the extent that no significant shortening will take place under load.

3.8.5 *Backshoring precautions*—Stripping forms before slabs are strong enough to carry their own dead load and construction loads above (if applicable) requires knowledgeable supervision and extreme caution. Care must be exercised to insure that individual shores are not overloaded during stripping. The following procedure should be followed during stripping and backshoring beam and girder construction. *The procedure does not apply to reshoring which requires that structural members be strong enough to support their own weight before stripping the formwork.*

The forms should be removed in such a manner that the individual structural members are not allowed to deflect and carry load. Members with clear spans of 10 ft or more should remain supported at approximately mid- or third-points as a minimum.

3.8.6 *Removal of reshoring or backshoring*—Shores should not be removed until the slab or member supported has attained sufficient strength to support all applied loads. Removal operations should be carried out in accordance with a planned sequence so that the structure supported is not subject to impact or loading eccentricities.

3.8.7 *Post-tensioning effects on shoring and reshoring*—The design and placement of shores, reshores, and backshores for post-tensioned construction requires more consideration than for normal reinforced concrete. The stressing of post-tensioning steel can cause overloads to occur in shores, reshores, or backshores or other temporary supports. The stressing sequence appears to have the greatest effect. When a slab is post-tensioned, the force in the tendon produces downward load at the beam. If the beam is shored, the shoring must carry this added load. Magnitude of the load may approach the dead weight of the contributory area of the slab. If the floor slab is tensioned before the supporting beams and girders, a careful analysis of the load transfer to the beam or girder shores, reshores, or backshores will be required.

Similar load transfer problems occur in post-tensioned bridge construction.

CHAPTER 4—MATERIALS FOR FORMWORK

4.1—General

The selection of materials suitable for formwork should be based on maximum economy to the contractor, consistent with safety and the quality required in

Table 4.2—Form materials with data sources* for design and specification

Item	Principal use	Reference data
Lumber	Form framing, sheathing, and shoring	"American Softwood Lumber," PS20 National Design Specification for Wood Construction (NFPA)
		Wood Handbook: Wood as an Engineering Material, Reference 4.3
		Wood Structural Design Data, Reference 4.4 *Wood Engineering*, Reference 4.5
		Timber Construction Manual, Reference 4.6
		"Code for Engineering Design in Wood" (Canada), CAN3-086
		"Concrete Forms," Reference 4.7
Plywood	Form sheathing and panels	"Construction and Industrial Plywood," PS1
		"Concrete Forming," Reference 4.8
		"Plywood Design Specification," APA
Steel	Panel framing and bracing	*Manual of Steel Construction*, Reference 4.9
	Heavy forms and falsework	*Cold-Formed Steel Design Manual*, Reference 4.10
	Column and joist forms	"Forms for One-Way Concrete Joist Construction," ANSI A48.1
		"Forms for Two-Way Concrete Joist Construction," ANSI A48.2
		"Code of Standard Practice for Concrete Joist Construction," part of Reference 4.1
	Stay-in-place forms	ASTM A 464.1 (galvanized steel)
Aluminum†	Lightweight panels and framing; bracing and horizontal shoring	*Aluminum Construction Manual*, Reference 4.11
Reconstituted wood panel products‡	Form liner and sheathing	"Mat-Formed Wood Particle-board," ANSI A208.1
		"Hardboard Concrete Form Liners," LLB-810a
Insulating board, wood or glass fiber	Stay-in-place liners or sheathing	ASTM C 532 (insulating formboard)
Fiber or laminated paper pressed tubes or forms	Column and beam forms; void forms for slabs, beams, girders, and precast piles	
Corrugated cardboard	Internal and under-slab voids; voids in beams and girders (normally used with internal "egg crate" stiffeners)	"A Study of Cardboard Voids for Prestressed Concrete Box Slabs," Reference 4.12
Concrete	Footings, stay-in-place forms, molds for precast units	ACI 318
		"Precast Concrete Units Used as Forms for Cast-in-Place Concrete," ACI 347.1R
Fiberglass-reinforced plastic	Ready-made column and dome pan forms; custom-made forms for special architectural effects	"Reinforced Plastic Forms for Concrete," Reference 4.14
		Plastic Laminate Materials, Their Properties and Usage, Reference 4.13
Cellular plastics	Form lining and insulation; permanent forms	"Cellular Plastics in Construction," Reference 4.15
		"Cellular Plastics for Building," Reference 4.16
Other plastics: polystyrene, polyethylene, polyvinyl chloride	Form liners for decorative concrete	
Rubber	Form lining and void forms	
Form ties, anchors and hangers	For securing formwork against placing loads and pressures	See Section 2.4 for recommended safety factors
Plaster	Waste molds for architectural concrete	
Coatings	Facilitate form removal	
Steel joists	Formwork support	"Standard Specifications and Load Tables for Open Web Steel Joists," Reference 4.17
Steel frame shoring	Formwork support	"Recommended Horizontal Shoring Beam Erection Procedure," Reference 4.18
		"Recommended Safety Requirements for Shoring Concrete Formwork," Reference 4.19
		Design Manual for Structural Tubing, Reference 4.20
Form insulation	Cold weather protection of concrete	ACI 306-R; see also *"Cellular Plastics"*

*In addition to ACI Special Publication No. 4, *Formwork for Concrete*. Handbooks, standards, and specifications cited here are listed either in Chapter 4 references or Chapter 8.
†Shall be readily weldable, nonreactive to concrete or concrete containing calcium chloride, and protected against galvanic action at points of contact with steel.
‡Check surface reaction with wet concrete.

the finished work. Approval by the engineer/architect, if required, should be based only on safety and quality of finished work. Where concrete surface aesthetics are critical, the architect/engineer should make provision for preconstruction mock-ups. See Chapter 5 for architectural concrete provisions.

4.2—Properties of materials

4.2.1 *General—Formwork for Concrete*[1.1] describes the formwork materials commonly used in the United States and provides extensive related data for form design. Much useful specification and design information is also available from manufacturers and suppliers of materials. Table 4.2 indicates other specific sources of design and specification data for formwork materials. This tabulated information should not be interpreted to exclude the use of any other materials which can meet quality and safety requirements established for the finished work.

4.2.2 *Sheathing*—Sheathing is the supporting layer of formwork closest to the concrete. It may be in direct contact with the concrete or be separated from it by a form liner. Sheathing consists of wood, plywood, metal, or other materials capable of transferring the load of the concrete to supporting members such as joists or studs.

In selecting and using these materials, important considerations are: (1) strength; (2) stiffness; (3) release; (4) reuse and cost per use; (5) surface characteristics imparted to the concrete such as wood grain transfer, gloss, paintability; (6) resistance to mechanical damage, such as from vibrators and abrasion from slipforming; (7) workability for cutting, drilling, and attaching fasteners; (8) adaptability to weather and extreme field conditions, temperature, and moisture; and (10) weight and ease of handling.

4.2.3 *Structural supports*—Structural support systems carry the sheathing. Important considerations are: (1) strength; (2) stiffness; (3) dimensional accuracy and stability; (4) workability for cutting, drilling, and attaching fasteners; (5) weight; (6) cost and durability.

4.3—Accessories

4.3.1 *Form ties*—A form tie is a tensile unit adapted to holding concrete forms against the active pressure of freshly placed plastic concrete. In general, it consists of an inside tensile member and an external holding device, both made to specifications of various manufacturers. These manufacturers also publish recommended working loads on the ties for use in form design. There are two basic types of tie rods, the prefabricated rod or band type, and the threaded internal disconnecting type. Their suggested working loads range from 1000 to over 50,000 lb.

4.3.2 *Form anchors*—Form anchors are devices used to secure formwork to previously placed concrete of adequate strength. The devices normally are embedded in the concrete during placement. Actual load-carrying capacity of the anchors depends on their shape and material, the strength and type of concrete in which they are embedded, the area of contact between concrete and anchor, and the depth of embedment and location in the member. Manufacturers publish design data and test information to assist in the selection of proper form anchor devices.

4.3.3 *Form hangers*—Form hangers are devices used to support formwork loads from a structural steel or precast concrete framework.

4.3.4 *Side form spacers*—A side form spacer is a device that maintains the desired distance between a vertical form and reinforcing bars. Both factory-made and job-site fabricated devices have been successfully used. Advantages and disadvantages of the several types are explained in References 1.3, 4.1, and 4.2.

4.3.5 *Recommendations*

4.3.5.1 Recommended factors of safety for ties, anchors, and hangers are given in Section 2.4. Yield point of the material should not be exceeded.

4.3.5.2 The rod or band type form tie, with supplemental provision for spreading the forms and a holding device engaging the exterior of the form, is the common type used for light construction.

The threaded internal disconnecting type is more often used for formwork on heavy construction such as heavy foundations, bridges, power houses, locks, dams, and architectural concrete.

Removable portions should be of a type which can be readily removed without damage to the concrete and which leave the smallest practicable holes to be filled. Removable portions of the tie should be removed unless the contract documents permit their remaining in place.

A minimum specification for form ties should require that the bearing area of external holding devices be adequate to prevent excessive bearing stress in form lumber.

4.3.5.3 Form hangers must support the dead load of forms, weight of concrete, and construction and impact loads. Form hangers should be symmetrically arranged on the supporting member to minimize twisting or rotation of supporting members.

4.3.5.4 Where the concrete surface is to be exposed and appearance is important, the proper type of form tie or hanger that will not leave exposed metal at the surface is essential. Otherwise, noncorrosive materials should be used when tie holes are left unpatched, exposing the tie to the elements.

4.4—Form coatings and release agents

4.4.1 *Coatings*—Form coatings or sealers are usually applied in liquid form to contact surfaces either during manufacture or in the field to serve one or more of the following purposes:

a. Alter the texture of the contact surface.

b. Improve the durablity of the contact surface.

c. To facilitate release from concrete during stripping.

d. Seal the contact surface from intrusion of moisture.

4.4.2 *Release agents*—Form release agents are applied to the form contact surfaces to prevent bond and thus facilitate stripping. They may be applied permanently to form materials in manufacture or applied to the form before each use. When applying in the field, be careful to avoid coating adjacent construction joint surfaces or reinforcing steel.

4.4.3 *Manufacturers' recommendations*—Manufacturers' recommendations should be followed in the use of coatings, sealers, and release agents, but independent investigation of their performance is recommended before use. Where surface treatments such as paint, tile adhesive, sealers, or other coatings are to be applied to formed concrete surfaces, be sure that adhesion of such surface treatments will not be impaired or prevented by use of the coating, sealers, or release agent. Also, bonding of subsequent concrete placements must be considered.

4.5—References

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4.3. *Wood Handbook: Wood as an Engineering Material*, Agriculture Handbook No. 72, Forest Products Laboratory, U.S. Department of Agriculture, 1974, 421 pp. (Available from U.S. Government Printing Office, Washington, D.C.).

4.4. *Wood Structural Design Data*, National Forest Products Association, Washington, D.C., 1978, 236 pp.

4.5. Gurfinkel, German, *Wood Engineering*, Kendall Hunt Publishing Co., Dubuque, 1981, 552 pp.

4.6. American Institute of Timber Construction, *Timber Construction Manual*, 3rd Edition, John Wiley & Sons, New York, 1986, 836 pp.

4.7. "Concrete Forms," Western Wood Products Association, Portland, Oregon.

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4.9. *Manual of Steel Construction*, 8th Edition, American Institute of Steel Construction, Chicago, 1980, 804 pp.

4.10. *Cold-Formed Steel Design Manual*, American Iron and Steel Institute, New York.

4.11. *Aluminum Construction Manual*, Aluminum Association, New York.

4.12. Ziverts, George J., "A Study of Cardboard Voids for Prestressed Concrete Box Slabs," *Journal*, Prestressed Concrete Institute, V. 9, No. 3, June 1964, pp. 66-93, and V. 9, No. 4, Aug. 1964, pp. 33-68.

4.13. Ziverts, George J., "Reinforced Plastic Forms for Concrete," *Proceedings*, 21st Annual Meeting, Society of the Plastics Industry, New York, 1966.

4.14. Beach, Norman E., *Plastic Laminate Materials, Their Properties and Usage*, Foster Publishing Co., Long Beach, Calif.

4.15. "Cellular Plastics in Construction," Building Materials Committee, Cellular Plastics Division, Society of the Plastics Industry, New York.

4.16. Cellular Plastics for Building," *Digests* 93 and 94, Building Research Station, Garston, Watford, May and June 1968. (Available from Her Majesty's Stationery Office, London).

4.17. "Standard Specifications and Load Tables for Open Web Steel Joists," Steel Joist Institute, Arlington, Va.

4.18. "Recommended Horizontal Shoring Beam Erection Procedure," Scaffolding, Shoring, and Forming Institute, Cleveland.

4.19. "Recommended Safety Requirements for Shoring Concrete Formwork," Scaffolding, Shoring, and Forming Institute, Cleveland.

4.20. *Design Manual for Structural Tubing*, Committee of Steel Pipe Producers, American Iron and Steel Institute, New York, 1974, 111 pp.

CHAPTER 5—ARCHITECTURAL CONCRETE

5.1—Introduction

5.1.1 *Objective*—General requirements for formwork presented in preceding chapters for the most part also apply to architectural concrete. Additional information is available in ACI 301R.

This chapter identifies and emphasizes additional factors that may have a critical influence on formwork for cast-in-place architectural concrete. Tilt-up and other types of precast architectural concrete are not considered here.

5.1.2 *Definition*—Architectural concrete is defined as concrete that is exposed as an interior or exterior surface in the completed structure, definitely contributes to its visual character, and is specially designated as such in the contract documents. Particular care must be taken in the selection of materials for and in the design and construction of the formwork, as well as in the placing and consolidation of such concrete, to eliminate bulges, offsets, or other unsightly features in the finished surface and to maintain the integrity of the surface texture or configuration. The character of the concrete surface to be produced must also be considered when the form materials are selected. Special attention should be given to closure techniques, concealment of joints in formwork materials, and to the sealing of forms to make them watertight.

5.1.3 *Factors in addition to formwork*—Many factors other than formwork affect the architectural effects achieved in concrete surfaces. They start at the design stage and carry through to the completed project. Factors affecting the concrete can also include the mix design or aggregate, the method of placing the concrete, and the consolidation technique. Chemicals may have an effect on the final product, whether used as additives in the mix; applied directly to the concrete, such as curing compounds; or applied indirectly, such as form release agents. Even after the structure is completed, weather and air pollution will affect the appearance of the concrete. These as well as other influencing factors must be identified and their effects evaluated during the initial design stages. However, the single most important factor for success of an architectural concrete job is good workmanship.

5.1.4 *Uniform construction procedures*—A major objective of architectural concrete is to obtain uniformity of color and surface finish. The best way for the contractor to achieve this uniformity is to be consistent in all construction practices. Forming materials must be kept the same, and release agents must be applied uniformly and consistently. Placement and consolidation of the concrete should be standardized so that uniform density is achieved. Stripping and curing sequences must be held constant throughout the work to control color variations.

5.2—Role of the architect

5.2.1 *Preplanning*—Much architectural concrete is also structural, but the quality of surface generally desired for architectural concrete is of a different level from that which is satisfactory for structural concrete, and is therefore more costly. The architect who keeps abreast of the state of the art in forming and concrete technology can use this information during the design process to keep his plans in line with the budget for the structure. Intricacies and irregularities may be costly far out of proportion to their esthetic contribution. For economy, the architect can make form reuse possible by standardizing building elements such as columns, beams, windows, and by making uninterrupted form areas the same size wherever possible to facilitate use of standard form gangs or modules. Increased size of these uninterrupted areas will contribute to forming economy. A prebid conference with qualified contractors will bring out many practical considerations before the design is finalized.

5.2.2 *Contract documents and advance approvals*—The architect should prepare contract documents that fully instruct the bidder as to the location and desired appearance of architectural surfaces, as well as other specific requirements listed in Sections 5.2.3 through 5.2.7. On major work this is frequently achieved by specifying a preconstruction mockup prepared and finished by the contractor for approval by the architect, using proposed form materials, jointing techniques, and form surface treatments, such as wetting, oiling, or lacquering. Once such a mockup has been completed to the satisfaction of the architect, it remains at the site for the duration of the work as a standard with which the rest of the work must comply.

Design reference samples—smaller specimens of concrete with the proposed surface appearance—may also be created for approval of the architect. Samples like these, kept at the job site for reference, are not as good as a full-scale mockup but may be helpful. Samples should be large enough to adequately represent the surface of the concrete. If the samples are to be used as a basis for acceptance, several should be made to represent the variation that may occur in the finish.

In the absence of physical mockups or reference samples, it may be helpful to specify viewing conditions under which the concrete surfaces will be evaluated for compliance with the specifications.

5.2.3 *Tolerances*—The architect should specify dimensional tolerances considered essential to successful execution of the design. ACI 117 can be consulted, but the architect must realize that the tolerances therein are for concrete construction in general, and more restrictive tolerances may be required for architectural work. No numerical limits are suggested here since the texture, lighting, and configuration of surfaces will all have an influence.

5.2.4 *Camber*—The builder can be expected to camber forms to compensate for deflection of the forms themselves during construction. However, the architect must specify any additional camber required to compensate for structural deflection or optical sag (the illusion that a perfectly horizontal long span member is sagging). The architect should be aware that it is customary to check horizontal members for compliance with tolerances before removal of forms and shores.

5.2.5 *Joints and details*—Location, number, and details of such items as openings, contraction joints, construction joints, and expansion joints should be shown on the design drawings or the architect should specify a review of the proposed location of all of these details as shown on the formwork drawings.* Since it is impossible to disguise the presence of joints in the form face, it is important for their positions to be predetermined and if possible planned as part of the architectural effect.

The architect can plan joint locations between surface areas on a scale and module suitable to the size of available materials and prevailing construction practices. If this is not esthetically satisfactory, dummy joints can be introduced to give a smaller pattern. Actual joints between sheathing materials can be masked by means of rustication strips (splayed fillets) attached to the form face. Rustication strips at horizontal and vertical construction joints can also create crisp edges accented by shadow lines instead of the potential ragged edge of a construction joint left exposed to full view.

Sometimes construction joints in beams can be concealed above the support columns, and joints in floors above their supporting beams instead of in the more customary regions of low shear.

5.2.6 *Ties and inserts*—Form ties and accompanying tie holes are an almost inescapable part of wall surfaces. Recognizing this, architects frequently integrate tie holes into the visual design quality of the surface. If this is planned and any effects or materials other than those provided in Section 5.3.4 are desired, they should be clearly specified as to both location and type.

Where tie holes are to be patched or filled, the architect should specify the treatment desired unless it has been shown on the preconstruction mockup.

5.2.7 *Cover over reinforcing steel*—Adequate cover over reinforcement as required by codes is needed for protection of steel and long-term durability of the concrete. Proper reinforcement properly located is important in control of surface cracking. For positive assurance of maintaining required cover, it is recommended that the architect specify appropriate side form spacers as defined in Section 4.3.4.

There is no advantage in specifying more cover than required by code, since excessive cover can permit increased cracking. However, the architect must specify sufficient cover to allow for any reduction that will result from incorporation of grooves or indented details and from surface treatments such as aggregate exposure and tooling. The maximum thickness of any material to be removed should be added to basic required cover.

*Some guidance on joint locations can be found in ACI 224R, 303R, and 332.

5.3—Materials and accessories

5.3.1 *Sheathing or form facing*—Architectural concrete form sheathing must be of appropriate quality to maintain uniformity of concrete surfaces through multiple uses, and to control deflection within appropriate limits. Plywood, steel, glass-fiber-reinforced plastic, and aluminum may all be suitable as sheathing or facing materials. Select the grade or class of material needed for pressure, framing, and deflection requirements. Be sure that the chosen material meets the specification requirements for concrete surface texture. Procedures for controlling rusting of steel must be carefully followed.

5.3.2 *Structural framing*—Form facing can be supported with lumber, steel, or aluminum members straight and rigid enough to meet the architectural specifications.

5.3.3 *Form liners*—A form liner is a material, not structurally required, attached to the inside face of the form to alter or improve surface texture or quality of the concrete. Wood, rigid plastic, elastomeric materials, and glass-fiber-reinforced plastics are all suitable liner materials when carefully detailed and fabricated. Plastics must be handled and assembled with care to avoid distortion caused by daily temperature cycles at the job site.

5.3.4 *Form ties*—Form tie assemblies for architectural concrete should permit tightening of forms and be of such type as to leave no metal closer to the surface than 1½ in. for steel ties and 1 in. for stainless steel ties. They should not be fitted with lugs, cones, washers, or other devices that will leave depressions in the concrete less than the diameter of the device unless otherwise specified. Ties should be tight fitting or holes sealed to prevent leakage at the holes in the form. If textured surfaces are to be formed, ties should be carefully evaluated as to fit, pattern, grout leakage, and esthetics.

5.3.5 *Side form spacers*—Side form spacers, as defined in Section 4.3.4, are particularly important in architectural concrete to maintain adequate cover over reinforcing steel and prevent development of rust streaking on concrete surfaces. Plastic, plastic-protected, rubber-tipped, or other noncorroding spacers must be attached to the reinforcing bar so that they do not become dislodged during concrete placement and vibration. The number and location of the side form spacers must be adequate for job conditions. However, they should never be more than 6 ft on centers, and always staggered.

5.4—Design

5.4.1 *Special considerations*—The general procedure will follow principles outlined in Chapter 2. However, the form designer will frequently have limitations imposed by the architectural design. Some of these considerations are: tie spacing and size, form facing preferences, location and special treatment of form joints, special tolerances, and use of admixtures. Since these factors can influence form design, they must be fully reviewed at the beginning.

5.4.2 *Lateral pressure of concrete*—Architectural concrete may be subjected to external vibration, revibration, set retardants, superplasticizers, and slumps greater than those assumed for determining the lateral pressure as noted in Section 2.2.2. Particular care must be exercised in these cases to design the forms for the increased lateral pressures arising from the aforementioned sources as noted in Section 2.2.2.

5.4.3 *Structural considerations*—Since deflections in the contact surface of the formwork reflect directly in finished surfaces under varying light conditions, forms for architectural concrete must be designed carefully to minimize deflections. Deflections may govern design rather than bending (flexural stress) or horizontal shear. Deflections of sheathing, studs, and wales should be designed so that the finished surface meets the architectural specifications. Wood forms bow with reuse, and hence more bulging will be reflected in the surface formed after several uses. This effect should be considered when designing wood forms.

When tie size and spacing are limited by the architect, the form designer may have to reverse the usual procedure to arrive at a balanced form design. Given the capacity of the available tie and the area it supports, he can find the allowable pressure, design supporting members, and establish a rate of placing.

Where wood forms are used, stress-graded lumber (or equivalent) free of twists and warps should be used for structural members. Form material should be sized and positioned to prevent deflections detrimental to the surfaces formed. Joints of sheathing materials should be backed with structural members to prevent offsets.

5.4.4 *Tie and re-anchor design*—Tie layout should be planned. If the holes are to be exposed as part of the architectural concrete, tie placement should be symmetrical with the member formed. If tie holes are not to be exposed, ties should be located at rustication marks, control joints, or other points where the visual effect will be minimized.

Externally braced forms may be used instead of any of the above mentioned methods to avoid objectionable blemishes in the finished surface. However, externally braced forms may be more difficult and more costly to build.

Consideration should be given to re-anchoring forms into preceding or adjacent pours to achieve a tight fit and prevent grout leakage at these points. Ties should be located within 18 in. of the construction joint wherever possible to facilitate re-anchoring the form to adjacent pours. Sheathing should not overlap the adjacent pour by more than 1½ in.

5.4.5 *Joints and details*—In architectural concrete, joints should, where feasible, be located at the junction of the formwork panels. At contraction or construction joints, rustication strips should be provided and fastened to the face of forms.

Corners should be carefully detailed to prevent grout leakage. Sharp corners should, wherever possible, be

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eliminated by the use of chamfer strips.

5.4.6 *Tolerances*—The form designer must check for specified dimensional tolerances that may have a bearing on what deflections can be permitted when designing the forms. If no special tolerances are given, the form designer may use ACI 117 tolerances for structural concrete.

5.5—Construction

5.5.1 *General*—Forms should be carefully built to resist the pressures to which they will be subjected and to limit deflections to a practicable minimum within the tolerances specified.

Joints in structural members should be kept to a minimum, and where necessary, should be suitably spliced or otherwise constructed so as to maintain continuity.

Pour pockets for vibrating or placing concrete should be planned to facilitate careful placement and consolidation of the concrete to prevent segregation, honeycomb, sanding, or cold joints in the concrete.

Attachment of inserts, rustication strips, ornamental reliefs, etc., should be planned so that forms may be removed without exerting pressure on these attachments.

Where special forming systems are specified by the engineer for structural purposes (such as one-way and two-way joist systems) in areas that are considered architectural, the architect and engineer should coordinate their requirements to be sure the architectural effect is consistent to be sure the architectural effect is consistent with the forming method and material specified.

Forms which are to be reused should be carefully inspected after each use to assure that they have not become damaged, distorted, disassembled, or otherwise unable to perform as designed.

5.5.2 *Sheathing and jointing*—Contact surfaces of the formwork should be carefully installed to produce neat and symmetrical joint patterns unless otherwise specified. Joints should be either vertical or horizontal and, where possible, should be staggered so as to maintain structural continuity.

Nailing should be done with care using hammers with smooth and well-dressed heads to prevent marring of the form surfaces. Box nails should be used when required on the contact surface and should be placed in a neat pattern.

Wherever possible, sheathing or panel joints should be positioned at rustication strips or other embedded features which may conceal or minimize the joint.

Where construction joints are necessary, they should be formed with a grade strip attached to the form to define a clean straight line on the joint of the formed surface. Formwork should be tightened at a construction joint before the next placement to prevent seepage of water between the form and previously placed concrete surfaces.

Architectural concrete forms should be leakproof. One method to prevent loss of water from the concrete at the joints between sections of the formwork and at construction joints is to attach a gasket of flexible material to the edge of each panel. The gasket is compressed when the formwork is assembled or placed against the existing concrete.

Textured surfaces on multi-lift construction should be separated with rustication strips or broad reveals because accumulation of construction tolerances and/or random textures prevent texture matching. Furthermore, the grout seal between the bottom of a textured liner and the top of the previous pour is impractical without the rustication strip.

5.5.3 *Cleaning, coating, and release agents*—Form coatings or releasing agents should be applied before reinforcing steel is placed and should be applied carefully to avoid contacting adjacent construction joints or reinforcing. No form coating should be used unless it can be guaranteed not to stain the concrete or impair the adhesion of paints or other intended surface treatments.

Form scalers should be tested to assure that they will not adversely affect the texture of the form lining material.

Ties that are to be pulled from the wall must be coated with nonstaining bond breaker or encased in sleeves to facilitate removal.

Forms should be carefully cleaned and repaired between uses to prevent deterioration of the quality of surface formed. Film or splatter of hardened concrete should be thoroughly removed.

5.5.4 *Ornamental liners and detail*—Ornamental concrete usually is formed by elastomeric molds or wood, plastic, or plaster waste molds. Members making up wood molds should be kerfed on the back wherever such members may become wedged between projections in the ornament. Molds must be so constructed that joints will not be opened by slight movement or swelling of the wood. Joints in the molds should be made inconspicuous by pointing.

The molds should be carefully set in the forms and securely held in position to reproduce the design shown on the drawings. Where wood forms adjoin molds, the wood should be neatly fitted to the profile of the mold and *all joints* should be carefully pointed. The molds and the adjacent wood forms should be so detailed that the wood forms can be stripped without disturbing the molds. A slight draft on the edge of molds or pattern strips should be provided to permit removing the detail material without damaging the concrete. Special provisions should be made for early form removal and/or retardation when sandblasting, wire brushing, or other treatments are required.

Form liners should be attached securely with fasteners or glue recommended by the manufacturer. The form behind the liner should be sound to hold the fasteners. When gluing, the surfaces should be cleaned and dried thoroughly so that the glue will bond. Do not use glue at temperatures lower than those recommended by the manufacturer.

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5.6—Form removal

5.6.1 *Avoiding damage*—When concrete surfaces are to be left as cast, it is important not to damage or scar the concrete face during stripping. Forms should be supported so that they do not fall back or against the architectural surface. The use of pry bars and other stripping tools should be strictly supervised. In no case should pry bars be placed directly against the concrete. Even the use of wood or plastic wedges does not insure that damage will not occur.

Once formwork is removed, the architectural surfaces must be protected from continuing construction operations.

5.6.2 *Concrete strength*—It is desirable for architectural concrete to have a higher compressive strength than normal for stripping. This can be accomplished by adjusting the mix proportions or leaving forms in place longer. If concrete is not strong enough to overcome the adhesion between the form surface and the concrete, concrete may scale or spall. Thus, a good quality surface might require the forms to stay in place longer. However, the longer the forms stay in place, the darker the concrete will become. The architect/engineer should specify what concrete strength is required before stripping can take place.

5.6.3 *Uniformity*—To insure surface quality, uniformity in stripping time and curing practices is essential. Where the objective is to produce as consistent an appearance as possible, it will be beneficial to protect the concrete during its early life by leaving the formwork in place somewhat longer than normal. Early exposure of concrete to the air affects the manner in which the surface dries. The ambient conditions can thus influence the eventual color of the concrete.

5.6.4 *Avoiding thermal shock*—Cold weather concreting requires that special attention be paid to the sudden temperature change of concrete. To avoid thermal shock and consequent crazing of the concrete surface, the change in temperature of the concrete should be controlled within the limits outlined in ACI 303R. This can be accomplished by heating the work area, leaving the forms in place in order to contain the heat of hydration, or by insulating the concrete after the forms have been removed (see ACI 306R).

CHAPTER 6—SPECIAL STRUCTURES

6.1—Discussion

In general, formwork for all structures should be designed, constructed, and maintained in accordance with recommendations in Chapters 1 through 4. This section deals with the additional requirements for formwork for several special classes of work. Attention is directed to ACI 344R for information on design and construction of circular prestressed concrete structures.

6.2—Bridges and viaducts, including high piers

6.2.1 *Discussion*—For bridges, the construction and removal of formwork must be planned in advance. Forms and supports should be sufficiently rigid to assure that the finished structure will fulfill its intended structural function and that exposed concrete finishes will present a pleasing appearance to the public.

6.2.2 *Shoring and centering*—Follow recommended practice in Sections 3.5 and 3.7 for erection and removal. In continuous structures, support should not be released in any span until the first and second adjoining spans on each side have reached the specified strength.

6.2.3 *Forms*—Forms may be of any of a large number of materials but most commonly wood or metal. They must be built mortar-tight of sound material sufficiently strong to prevent distortion during placing and curing of the concrete.

6.3—Structures designed for composite action

6.3.1 *Recommendations*—Structures or members that are designed so that the concrete portions act compositely with other materials or with other parts of the structure present special problems of forming which should be anticipated in the design of the structure. Requirements for shoring or other deflection control of the formwork should be clearly presented by the engineer/architect in the specifications. Where successive placements are to act compositely in the completed structure, deflection control becomes extremely critical.

Shoring, with or without cambering of portions of the structure during placement and curing of the concrete, should be analyzed separately for the effects of dead load of newly placed concrete and for the effect of other construction loads that may be imposed before the concrete attains its design strength.

6.3.2 *Design*—Formwork members and shores should be designed to limit deflections to a practical minimum consistent with the structural member being constructed.

Where camber is specified for previously installed components of the structure, allowance should be made for the resultant preloading of the shores before application of the dead load of concrete.

In members constructed in several successive placements, such as box girder structures, formwork components should be sized, positioned, and/or supported to minimize progressive increases in deflection of the structure which would excessively preload the reinforcing steel or other portions of the composite member.

In multistory work where shoring of composite members is required, consideration should be given to the number of stories of shores necessary, in conjunction with the speed of construction and concrete strengths, to minimize deflections due to successive loadings. Distinction should be made in such analyses for shores posted to relatively unyielding support such as foundations instead of to structures or members already in elastic support (see Section 3.8).

Composite construction may have beams of relatively light cross section that are fully adequate when construction is complete. However, during construction these beams may not be laterally supported by the formwork, leaving them with a high slenderness ratio

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and reduced beam strength. The engineer/architect should alert the contractor to this problem in general notes on the structural drawings or in notes on applicable drawings when this condition exists. The form designer should be alert to this possibility and provide shoring or lateral support where needed.

6.3.3 *Erection*—Construction and/or erection of formwork for composite construction follows basic recommendations contained in Chapter 3. Shoring of members that will act compositely with the concrete to be placed should be done with great care to assure sufficient bearing, rigidity, and tightness so as to prevent settlement or deflections beyond allowable limits. Wedges, shims, jacks, etc., should be provided to permit adjustment if required before or during concreting as well as to permit removal without jarring or impact of the completed construction. Provision should be made for readily checking the accuracy of position and grade during placement. Even though adjustment of forms may be possible during or after the pour, it is not recommended. Any required adjustment should be made prior to initial set of the concrete.

Where camber is required, distinction should be made between that part which is an allowance for settlement or deflection of formwork or shoring and that which is provided for design loadings. The former should generally be the responsibility of the contractor who designs the forms and supports unless such camber is stipulated by the engineer/architect. Measurement of camber provided for structural design loadings should be made after hardening of the concrete but before removal of the supports [see also Section 1.4.5(f)].

6.3.4 *Removal*—In addition to meeting the provisions of Section 3.7, forms and/or supports should be removed only after tests and specified curing operations indicate to the satisfaction of the engineer that the most recently placed concrete has attained the strength required to develop composite action, and then only after stated approval of the engineer/architect. The sequence of such removal should be approved by the engineer/architect.

6.4—Folded plates, thin shells, and long span roof structures

6.4.1 *Discussion*—For long span and space structures requiring a complex, three-dimensional design analysis and presenting three-dimensional problems in formwork design, erection, and removal, formwork planning should be done by engineers having the necessary special qualifications and experience. These engineers should consult and cooperate with the engineer/architect to make sure that the resulting surfaces will conform to his design.

6.4.2 *Design*

a. The engineer/architect should specify limiting values and directions of the reactive forces when the falsework is supported by the permanent structure.

b. When applicable, the engineer/architect should include a decentering sequence drawing with the bidding documents as a basis for the design of the forming and support system to be used by the contractor.

c. Lateral loads—In determining the lateral forces acting on the formwork, the wind load should be calculated on the basis of a minimum of 15 psf of projected vertical area as specified for wall forms in Section 2.2.3. For sturctures such as domes, negative forces due to suction created by the wind on the leeward side of the structure should be considered.

d. Analysis—The provisions of Sections 2.1.1 and 2.3 should be closely adhered to in such formwork planning.

Assumed design loads should be shown on the formwork drawings. Complete stress analyses should be prepared by competent structural engineers, and the maximum and minimum values of stress, including reversal of stress, should be shown for each member for the most severe loading conditions. Due regard should be given to unsymmetrical or eccentric loadings that might occur during concrete placement and during erection, decentering, or moving of traveler. The vertical or lateral deflection of the moving forms or travelers as well as the stability under various loads should be investigated to insure that the formwork will function satisfactorily and that the concrete tolerances will be met.

Particular care must be taken in the design and detailing of individual members and connections. Where trussed systems are used, connections must be designed to keep eccentricities as small as possible to minimize deflections of distortions.

Since the weight of the forms and falsework may, in many cases, be equal to or greater than the design live load of the structure, form details should be so designed as to avoid hanging up the form and falsework and thus overloading the structure itself during decentering.

e. Due to the special shapes involved, tolerances based on functions of these shapes should be specified by the engineer/architect in the bidding documents.

6.4.3 *Drawings*—When required, the contractor should submit detailed drawings of the formwork for approval of the engineer/architect.

These drawings should show the proposed concrete placing sequence and the resulting loads. To insure that the structure can assume its deflected shape without damage, the decentering or handling sequence of the formwork should be shown on the drawings.

Deflection of these structures may cause binding between the form and the concrete during decentering. Formwork drawings and form details must be planned to prevent binding and to facilitate stripping of forms. Drawings should show such details as type of inserts and joints in sheathing where spreading of the form may result in the form becoming keyed into the concrete.

6.4.4 *Approval*—The formwork drawings and procedures must comply with federal and local safety laws as well as with the contract drawings and specifications and meet the general requirements for formwork to assure the integrity and stability of the permanent structure itself. The engineer/architect should check the design and shop drawings for the formwork to insure that these requirements are met and approve them in writing.

6.4.5 *Construction*—In planning and erecting formwork, provision should be made for adequate means of adjustment during placing where necessary. Telltales should be installed to check alignment and grade during placement.

Where the forming system is based on a certain placing sequence, that sequence should be clearly defined and adhered to in the field.

6.4.6 *Removal of formwork*—Formwork should be removed and decentered in accordance with the procedure and sequence specified on the form drawings or on the contract documents. Decentering methods used should be planned to prevent any concentrated reaction on any part of the permanent structure. Due to the large deflections and the high dead load-to-live load ratio common to this type of structure, decentering and form removal should not be permitted until specified tests demonstrate that the concrete strength and the modulus of elasticity specified in contract documents have been reached. Moduli of elasticity may determine time of decentering although required compressive strengths may already have been attained. Generally, decentering should begin at points of maximum deflection and should progress toward points of minimum deflection, with the decentering of edge members proceeding simultaneously with the adjoining shell.

6.5—Mass concrete structures

6.5.1 *Discussion*—ACI 116R defines mass concrete as "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking." Mass concrete generally occurs in heavy civil engineering construction, such as in gravity dams, arch dams, gravity retaining walls, lock walls, power plant structures, and large building foundations. Special provisions usually are made to control the temperature rise in the mass by the use of cement or cementing material combinations possessing low or moderate heat-generating characteristics, by postcooling, cooling the fresh concrete, or by placing sequence.

Formwork for mass concrete falls into two distinct categories, namely, low and high lift. Low lift formwork, for heights 5 to 10 ft, usually consists of multiuse steel cantilever form units that incorporate their own scaffolding and, on occasion, lifting devices. High lift formwork is strictly comparable to the single-use wood forms used extensively for structural concrete.

6.5.2 *Lateral pressure of concrete*—The lateral pressure formulas for concrete placed in walls may be used for mass concrete. See Section 2.2.2.

Consideration should be given to placing sequence in the determination of pressure. Frequently, concrete is layered in such a way that a full liquid head is developed against the form on the closure end. In addition, the use of large concrete buckets may cause high impact loads near the forms.

6.5.3 *Design considerations*—Particular care must be taken to provide anchorage for forms with a batter and wall forms tied to a rock face. The ultimate strength of the tie rods must not exceed the ultimate strength of the anchor bar or bolt. The bending and welding of high tensile steel tie rods should be prohibited. Consideration should be given to form ties embedded in previously placed concrete to insure that such concrete has attained sufficient strength to sustain design loadings from the new placement as well as initial bolting stresses.

6.5.4 *Tolerances*—See Section 3.3 and ACI 117.

6.6—Underground structures

6.6.1 *Discussion*—Underground structures differ from corresponding surface installations in that the construction takes place inside an excavation instead of in the open, thereby providing unique problems in handling and supporting formwork and in the associated concrete placing. As a result, the following four factors usually make the design of formwork for underground structures entirely different than for their aboveground counterparts: First, concrete to fill otherwise inaccessible areas may be placed pneumatically or by positive displacement pump and pipeline; second, the rock sometimes is utilized as a form backing, thereby permitting the use of rock anchors and tie rods in lieu of external bracing and shores; third, the limits of the excavation demand special handling equipment that adds particular emphasis to the removal and reuse of forms; fourth, rock surfaces sometimes can be used for attaching hoisting devices.

When placement is by pneumatic or positive displacement pump and pipeline methods, the plastic concrete is forced, under pressure, into a void such as the crown of a tunnel lining. For more information on the pumping process, see ACI 304.2R.

6.6.2 *Design loads*

6.6.2.1 *Vertical loads*—Vertical and construction loads assumed in design of formwork for underground structures are similar to those for surface structures, with the exception of unusual vertical loads occurring near the crown of arch or tunnel forms and of flotation effect beneath tunnel forms.

In placing concrete in the crowns of tunnel forms, pressures up to 3000 psf have been induced in areas of overbreak and near vertical bulkheads from concrete placed pneumatically or by positive displacement pump. Until more definite recommendations can be made, the magnitude and distribution of pressure should be determined by the design engineer. In no case should the assumed pressure be less than 1000 psf acting normally to the form plus the dead weight of the concrete placed pneumatically or by pump.

6.6.2.2 *Lateral loads*—For shafts and exterior walls against rock, the values listed in Section 2.2.2 should apply.

When the shaft form relies on the single shear value of embedded anchors in the previous placement as a means of support, the minimum time lapse between successive placements (or minimum concrete strength) and maximum allowable loading additional to the dead weight of the form should be specified.

For arch forms and for the portions of tunnel forms above the maximum horizontal dimension or spring line of the form, the pressure should be compatible with the pressures discussed under vertical loads in Section 6.6.2.1

6.6.3 *Drawings*—In addition to the provisions of Chapters 1, 2, and 3, the following data should be included on the drawings for specialized formwork and formwork for tunnels:

6.6.3.1 All pressure diagrams used in the design of the form including diagrams for uplift, for unbalanced lateral or vertical loads, for pressurized concrete, or for any other load applicable to the particular installation.

6.6.3.2 Recommended method of supplemental strutting or bracing to be employed in areas where form pressures may exceed those just listed due to abnormal conditions.

6.6.3.3 Handling diagrams and procedures showing the proposed method of handling the form during erection or installation for concrete placement plus the method of bracing and anchorage during normal operation.

6.6.3.4 In the case of the tunnel arch form, whether it is intended for use with the unit or bulkhead system of concrete placement or is restricted to use with the continuously advancing slope method (see Section 6.6.4).

6.6.3.5 When placement of concrete by pumping or pneumatic methods is anticipated, the capacity and working pressure of the prime mover and the size, length, and maximum embedment of the discharge line should be as assumed in the design. Also, when the design provides for a method of placement other than by sustained pumping via a buried slick line, it should be clearly stated that the design pressures would be exceeded if sustained pumping were adopted.

6.6.4 *Construction*—The two basic methods of placing a tunnel arch entail problems in the construction of the formwork that require special provisions to permit proper reuse. These two basic methods are commonly known as the "bulkhead method" and the "continuously advancing slope" method.

The former is used exclusively where poor ground conditions exist, requiring the lining to be placed concurrently with tunnel driving operations. It is also used when some factor, such as the size of the tunnel, the introduction of reinforcing steel, or the location of construction joints precludes the advancing slope method. The advancing slope method, a continuous method of placement, usually is preferred for tunnels driven through competent rock, ranging between 10 and 25 ft in diameter and at least 1 mile in length.

The arch form for the bulkhead method is usually fabricated into a single unit between 50 and 150 ft long, which is stripped, moved ahead, and reerected using screw jacks or hydraulic rams. These are permanently attached to the form and supporting traveling gantry. The arch form for the continuously advancing slope method usually consists of eight or more sections that range between 15 and 30 ft in length. These are successively stripped or collapsed, telescoped through the other sections, and reerected using a form traveler.

Although the minimum stripping time for tunnel arch forms usually is established on the basis of experience, it can be safely predetermined by tests. It is recommended that at the start of a tunnel arch concreting operation, the minimum stripping time be 12 hr for exposed surfaces and 8 hr for construction joints. If the specifications provide for a reduced minimum stripping time based on site experience, such reductions should be in time increments of 30 min or less and should be established by laboratory tests and visual inspection and surface scratching of sample areas exposed by opening the form access covers. Arch forms should not be stripped prematurely when unvented ground water seepage could become trapped between the rock surface and the concrete lining.

6.6.5 *Materials*—The choice of materials for underground formwork usually is predicated on the shape, degree of reuse and mobility of the form, and the magnitude of pump or pneumatic pressures to which it is subjected. Usually, tunnel and shaft forms are made of steel, or a composite of wood and steel. Experience is of paramount importance in the design and fabrication of a satisfactory tunnel form, due to the nature of the pressures developed by the concrete, placing techniques, and the high degree of mobility usually required.

When reuse is not a factor, plywood and tongue-and-groove lumber sometimes are used for exposed surface finishes, but more consideration may be given to wood sheathing because the high humidity often precludes the normal shrinkage and warping.

CHAPTER 7—FORMWORK FOR SPECIAL METHODS OF CONSTRUCTION

7.1—Recommendations

The applicable provisions of Chapters 2, 3, and 4 also apply to the work covered in this chapter.

7.2—Preplaced aggregate concrete

7.2.1 *Discussion*—Preplaced aggregate concrete is made by injecting (intruding) mortar into the voids of a preplaced mass of clean, graded aggregate. For normal construction, the preplaced aggregates are wetted and kept wet until the injection of mortar into the voids is completed. In underwater construction, the mortar displaces the water and fills the voids. In both types of construction, this process can create a dense concrete having a high content of coarse aggregate.

The injected mortar contains water, fine sand, portland cement, pozzolan, and a chemical admixture designed to increase the penetration and pumpability of the mortar. The coarse aggregate is similar to coarse aggregate for conventional concrete. It is well-washed and graded from ½ in. to the largest size practicable. After compaction in the forms, it usually has a void content ranging from 35 to 45 percent. Refer to ACI 304.1R.

7.2.2 *Design considerations*—Due to the method of placement, the lateral pressures on formwork are considerably higher than those developed for conventional concrete as given in Section 2.2.2. The form designer should be alerted to the unique problems created by high-density concrete, by mass placings where heat of hydration and drying shrinkage are critical, and by differential pressures in the form structure when mortar injection varies greatly from one form face to another.* Because of the pressure created during aggregate packing and mortar pumping, forms must be anchored and braced far more securely than for ordinary concrete. Particular attention must be paid to uplift pressures created in battered forms. Provision must be made to prohibit even the slightest uplift of the form. Injection pipes spaced 5 to 6 ft apart, penetrating the face of the form, require that the form be checked for structural integrity as well as a means of plugging or shutting off the openings when the injection pipes are removed.

Forms, ties, and bracing should be designed for the sum of:

a. The lateral pressure of the coarse aggregate as determined from the equivalent fluid lateral pressure of the dry aggregate using the Rankine or Coulomb theories for granular materials; or a reliable bin action theory; and

b. The lateral pressure of the injected mortar; as an equivalent fluid the mortar normally weighs 130 lb per cu ft, but may weigh as much as 200 lb per cu ft for high-density mortars.

The time required for the initial set of the mortar (from 6 to 7 hr) and the rate of rise (1 to 6 ft per hr) should be ascertained. The maximum height of fluid to be assumed in determining the lateral pressure of the mortar is the product of the rate of rise (ft per hr) and the time of initial set in hours.

The lateral pressure for the design of formwork at any point is the sum of the pressures determined from Steps (a) and (b) for the given height.

7.2.3 *Construction*—In addition to the provisions of Chapter 3, the forms must be mortar-tight and effectively vented because preplaced aggregate concrete entails forcing mortar into the voids around the coarse aggregate.

The increased lateral pressure usually requires that the workmanship and details of formwork be of better quality than formwork for conventional concrete.

7.2.4 *Materials for formwork*—Tongue-and-groove lumber is preferred for exposed surfaces; the joints between boards permit the escape of traces of mortar. However, excessive bleeding will cause sand streaking, which will mar the appearance of the finished surface. When excessive bleeding is evident, caulking or sealing of the joints is recommended. For unexposed surfaces, mortar-tight forms of steel or plywood are acceptable. Prefabricated panel-type forms usually are not suitable because of the difficulty in making mortar-tight seals between panels. Absorptive form linings are not recommended because they permit the coarse aggregate to indent the lining and form an irregular surface. Form linings such as hardboard on common sheathing are not successful because they do not transmit the external form vibration normally employed for insuring a void-free finished surface.

7.3—Slipforms†

7.3.1 *Discussion*—Forming of concrete by the use of slipforms is similar to an extrusion process. Plastic concrete is placed in the forms, and the forms act as moving dies to shape the concrete. The rate of movement of the forms is regulated so that the forms leave the concrete only after it is strong enough to retain its shape while supporting its own weight and the lateral forces caused by wind and equipment. Formwork of this type can be used for vertical structures such as silos, storage bins, building cores, bearing wall buildings, piers, chimneys, shaft linings, communication and observation towers, nuclear shield walls and similar structures.

Horizontal slipforming lends itself to concrete structures such as tunnel linings, water conduits, drainage channels, precast elements, canal linings, highway median barriers, and paving.

Vertical slipforms are usually moved in small increments by jacks that propel themselves on smooth steel rods or tubing embedded in or attached to the hardened concrete. Horizontal slipforms generally move on a rail system, tractor treads, wheels, or a shaped berm. Working and storage decks and finisher's scaffolding are attached to and carried by the moving formwork.

The vertical or horizontal movement of forms may be a continuous process or a planned sequence of finite placements.

Slipforms used on such structures as tunnels and shafts should comply with the applicable provisions of Section 6.6. Slipforms used on mass concrete structures such as dams should comply with the applicable provisions of Section 6.5

7.3.2 *Vertical slipforms*

7.3.2.1 A vertical slipform system has five main components: sheathing; wales; yokes; jacks and jackrods; and working or storage decks and scaffolding.

The sheathing or vertical forms can be metal, glass-fiber-reinforced plastic, wood, or a combination of these materials. The function of the sheathing is to contain and shape the concrete.

Wales have three main functions:

1. They support and hold the sheathing in place.

*For additional information see ACI 359, ACI 207.1R, and *Concrete for Nuclear Reactors*, ACI Special Publication, No. 34.

†For silo construction refer to ACI 313.

2. They transmit the lifting force from the yokes to the sheathing.

3. They provide support for various platforms and scaffolding.

Yokes transmit the lifting forces from the jacks to the wales and resist the lateral force of plastic concrete within the form.

The jacks, by climbing up the jackrods, provide the force needed to raise the entire slipform system.

Various platforms, decks, and scaffolding complete the slipform system. They provide a space for storage of concrete, reinforcing steel, and embedments as well as serving as a working area for placing and finishing

7.3.2.2 *Design and construction considerations*—Slipforms should be designed and constructed and the sliding operation should be carried out under the immediate supervision of a person or persons experienced in slipform work.

7.3.2.2.1 *Sheathing*—should be a minimum of 3 ft 6 in. high* and should be constructed of at least a nominal 1-in. board (tongue and groove preferred), ¾-in. plywood with suitable bracing, 10-gage minimum steel sheets, or other approved materials. Caution should be exercised when using steel sheets in extremes of heat or cold due to their lack of insulating capability. Both plywood and boards should be designed with their face grain running vertical. The 1-in. boards should be spaced 1/16 to 1/8 in. apart to allow for expansion when they become wet. Soaking of all wood with a suitable form oil or waterproofing compound is desirable in order to reduce water absorption by sheathing.

Forms should be constructed with slight draft, in the range of 1/8 to ½ in., particularly for the inside faces so that the distance between inside form and outside form is greater at the bottom than at the top. This taper may be applied to the inside or both faces of the form. The true wall thickness is measured at the elevation where hardened concrete is maintained in the form. This elevation may vary from 1 to 2 ft above the bottom of the form. The taper can be established through use of batter strips placed on the inside face of the upper set of wales. The taper reduces friction resistance to movement of the forms during jacking.

7.3.2.2.2 Wales may be made of timber or metal. Two sets of wales are traditionally used, and they should be designed to hold the sheathing in place against lateral forces and friction forces, to transmit the lifting forces from the yokes to the form, and to support various scaffolding and decks.

Timber wales should be of 2- or 3-ply lumber at least one ply of which is nominal 2-in. material.

7.3.2.2.3 Lateral and diagonal bracing of forms must be provided to insure that the shape of the structure will not be distorted beyond allowable tolerances during the sliding operation.

7.3.2.2.4 The design of the yokes must provide for adequate clearance to install horizontal reinforcing bars and embedments in their correct locations prior to their submergence in the rising concrete. They should also be designed with a minimum of deflection to maintain the desired configuration and wall thickness. The spacing of the yokes must be planned with consideration for the reinforcing steel configuration, the location of openings, extreme loading points, and conditions of a similar nature.

7.3.2.2.5 A jacking system that provides for the simultaneous movement of the entire form in small increments of approximately 1 in. at 2- to 3-min intervals is recommended. This available jacking capacity may be desirable or necessary for short periods; forms do not usually maintain such speed of movement for extended times. Special care must be taken in choosing the capacity of the jacks and arranging them so that the forms will draw straight and true without strain or twist. Jackrods must be properly braced where not encased in concrete.

Due to the number of variables such as friction and unbalanced loads, it is essential that reserve jacking capacity be provided in the system.

7.3.2.2.6 Working decks are supported directly on the forms and rise with them. The deck must be designed to maintain the plan dimension throughout the height of the structure. Through the use of trusses, corner bracing, tie rods, and similar items this can be accomplished.

7.3.2.2.7 Drawings should be prepared by a competent and experienced engineer employed by the contractor, showing the jack layout, formwork, working decks, and scaffolds. A developed elevation of the structure should be prepared, showing location of all openings and embedments.

7.3.2.3 *Vertical loads*

7.3.2.3.1 In addition to the dead loads, live loads assumed for design of decks should not be less than the following:

Sheathing and joists	75 psf or concentrated buggy wheel loads, whichever is the greater
Beams, trusses, and wales	50 psf
Light-duty finishers' scaffolding	25 psf

7.3.2.3.2 The friction loads used in determining jacking requirements should normally be not less than 200 lb per linear ft of concrete wall when a nominal 3 ft 6-in. to 4-ft two-sided slipform is used.

7.3.2.3.3 Where working decks are used as a bottom form for cast-in-place construction, such as floor slabs or roof slabs, the deck must be designed for the dead load of the concrete plus any superimposed

*The minimum height is a function of the rate of slipping (ft per hour) and the time required for the concrete to gain sufficient strength to support itself without sagging after leaving the slipform. A slightly higher form will provide some working space in the top of the form for placing of concrete and reinforcement. Forms less than 3½ ft high are believed to be dangerously shallow. Forms as high as 5 ft may be required when low temperature or slow setting concrete is specified. Forms in excess of 6 ft high can be used in special applications such as piers and single-sided shaft slipforms.

loads, and in no case less than the design loads given in Section 2.2. Where the inside slipform becomes part of the slab system, fixing of the form to the concrete must be designed to withstand the vertical and lateral forces associated with placing of the slab.

7.3.2.3.4 Vertical loads and torsional forces resulting from deck loads and friction must also be considered. The forms must act as trusses for the vertical loads between jacking points. Knee braces or other appropriate support systems should be provided for top wales where span between yokes exceeds 6 ft or where vertical loads are unusually heavy.

7.3.2.4 *Lateral pressure of concrete*—The lateral pressure of fresh concrete to be used in designing forms, bracing, and wales may be calculated as follows (see Appendix for metric conversions of equation)

$$p = c_1 + \frac{6000R}{T}$$

where

c_1 = 100*
p = lateral pressure, psf
R = rate of concrete placement in ft per hr
T = temperature of concrete in the forms, deg F

Wales must be adequately nailed or bolted together to transmit shear due to lateral pressure of concrete, and vertical posts should be placed between wales at lift points.

7.3.2.5 *Tolerances*—Suggested tolerances for slipform construction are listed in ACI 117.

7.3.2.6 *Sliding operation*—Maximum rate of slide should be limited by the rate for which the forms are designed. In addition, both maximum and minimum rates of slide must be determined by an experienced slipform supervisor to accommodate changes in weather, concrete slump, initial set of concrete, and workability, and the many exigencies which arise during a slide and which cannot be predicted accurately beforehand. A person experienced in slipform construction must be present on the deck at all times during the slide operation.

During the initial placing of the concrete in slipform, the pour rate should not exceed that for which the form was designed.

The level of the hardened concrete in the form must be checked frequently by the use of a probe to establish safe lifting rates. Forms must be leveled before they are filled and must be maintained level unless otherwise required for out-of-tolerance corrections. Care must be taken to prevent drifting of the forms from alignment or designed dimensions and to prevent torsional movement.

*It is felt that c_1 = 100 is justified because vibration is slight in slipform work since the concrete is placed in shallow layers of 6 to 10 in. and because there is no revibration. However, for some applications such as gastight or containment structures, additional vibration may be required to achieve maximum density of the concrete. In such cases, the value of c_1 should be increased to 150.

Experience has shown that a plumb line, optical plummet, laser, or combination of these used in conjunction with a water level system is effective in maintaining the form on line and grade and for positioning openings and embedded items.

Alignment and plumbness of structure should be checked at least once during every 4 hr that the slide is in operation and preferably every 2 hr. In work that is done in separate intermittent slipping operations, a check of alignment and plumbness should be made at the beginning of each slipping operation.

More frequent readings should be taken on single tall structures with relatively small plan sections, as these structures tend to twist and go out of plumb more readily.

Sufficient plummeting should be provided to readily detect and evaluate movements of the form for all slipformed structures so that appropriate adjustment can be made by experienced personnel.

Detailed records of both vertical and lateral form movements should be maintained throughout the slipform operation.

7.3.3 *Horizontal slipforms*

7.3.3.1 *Design considerations*—For major structures, this specialized formwork should be designed by experienced, competent engineers employed or engaged by the contractor or form supplier. A complete structural analysis, including stress diagrams of the structural members, must be made to insure satisfactory performance. Due regard should be given to unsymmetrical and eccentric loadings and the fact that the machine must be regularly disassembled as it encounters siphons, bridges, chutes, etc. The large machines are usually hinged so that sections may be passed through or beneath structures. The vertical or lateral deflections, particularly of long-span machines, must be investigated, and sufficient rigidity provided to insure that concrete tolerances will be met. The stability of the machine under the aforementioned loading conditions must be carefully investigated to insure satisfactory performance.

7.3.3.2 *Drawings*—The general provisions of Section 2.1.4 should be met and the contractor should submit drawings of the slipform for review and approval by the engineer/architect. These drawings should show the handling diagrams, the placing procedure, and the provisions for insuring attainment of the required concrete surfaces.

7.4—Permanent forms

7.4.1 *Discussion*—Permanent forms, as the name implies, are forms left in place that may or may not become an integral part of the structural frame. These forms may be the rigid type such as metal deck, precast concrete, wood, plastics, and various types of fiberboard; or the flexible type such as reinforced water-repellent corrugated paper, or wire mesh with waterproof paper backing.

When the permanent form is used as a deck form, it is generally supported from the main structural frame with or without an intermediate system of temporary supports. If temporary supports are required under the structural frame members to support the weight of the fresh concrete without excessive deflection, such information should be specified by the engineer/architect.

7.4.2 *Design considerations*—If the permanent type form is not covered in the contract specifictions, (1) the manufacturer's specifications should be used; (2) the manufacturer's recommended practice* should be followed for size, span, fastenings, and other special features pertinent to this type of form, such as being water repellent and protected against chemical attack from wet concrete; and (3) the minimum requirements of Chapters 2 and 3 should be followed. Particular care should be taken in the design of such forms to minimize distortion or deformation of the form or supporting members under the construction loads.

The engineer/architect who specifies or permits the use of permanent rigid forms should consider in his structural analysis dead and live loads for the structure's intended usage, especially concentrated loads between supporting members.

When metal deck to become an integral part of the structure is used as a permanent form, its shape, depth gage, physical dimensions, and properties should be as called for in contract documents. If structural continuity is assumed in the design, the engineer should specify the required number of supports over which the form material should be continuou...

7.4.3 *Installation*

7.4.3.1 *Shop drawings*—The contractor should submit fully detailed shop drawings for all permanent deck forms to the engineer/architect for review and/or approval. Shop drawings should show all form thicknesses, metal gages, physical dimensions and properties, accessories, finishes, and methods of attachment to the various classes of the work.

7.4.3.2 *Fastenings*—The permanent deck form must be properly fastened to supporting members and to adjacent sections of form and properly lapped to provide a tight joint that will prevent loss of mortar during the placement of concrete. End closures for corrugated or fluted forms should be provided, where required, together with fill pieces where a tight fit is required. To prevent buckling, allowance should be made for expansion of metal deck forms.

Flexible types of forms (those that depend for lateral stiffness on supporting members) must be drawn tight for proper installation. Adequate temporary bracing or anchors must be provided in the plane of the top chord of the supporting members to prevent lateral buckling and rotation of these supports and to maintain the required tension in the flexible form.

Paper or metal forms used to form voids in concrete construction should be properly placed and anchored to reinforcement and to side or deck forms with wire ties or by other approved methods to prevent displacement or flotation during placing of concrete. End closures should be properly vented where necessary to eliminate cracking of concrete by reason of expansion of air in voids due to the heat of hydration of the concrete. Water should be prevented from entering voids. Where water intrusion is possible, weep holes should be provided to reduce its entrapment.

7.4.4 *Deflections*—The vertical and lateral deflections of the permanent form between supports under the load of fresh concrete should be investigated by the designer. Temporary supports should be used, if necessary, to keep deflection within desired tolerances.

7.5—Forms for prestressed concrete construction

7.5.1 *Discussion*—The engineer/architect should indicate in the contract documents any special requirements for prestressed construction.

It may be necessary to provide appropriate means of lowering or removing the formwork before full prestress is applied, to prevent damage due to upward deflection of resilient formwork.

Pretensioning or post-tensioning of strands, cables, or rods may be done with or without side forms of the member in place, in accordance with Section 7.5.2. Bottom forms and supporting shores or falsework must remain in place until the member is capable of supporting its dead load and anticipated construction loads, as well as any formwork carried by the member.

The concreting sequence for certain structures must also be planned so that concrete is not subjected to bending stress caused by deflection of the formwork.

7.5.2 *Design*

7.5.2.1 Where the side forms cannot be conveniently removed from the bottom or soffit forms after concrete has set, such forms should be designed for additional axial and/or bending loads which may be superimposed on them during the prestressing operation.

7.5.2.2 Side forms that must remain in place during the transfer of prestressing force should be designed to allow for vertical and horizontal movements of the cast member during the prestressing operation. The form should be designed to minimize restraint to elastic shortening in the prestressing operation. For example, plan small components or wrecking strips that can be removed or destroyed to relieve load on side forms as well as to eliminate their restraint during prestressing. In all cases the restraint to shrinkage of concrete should be kept to a minimum, and the deflections of members due to prestressing force and the elastic deformation of form or falsework should be considered in the design and removal of the forms.

7.5.2.3 Care should be exercised with post-tensioned slabs to assure that supporting shores do not fall out due to lifting of slab during tensioning. For large structures where the dead load of the member remains on the formwork during prestressing, displacement of the dead load toward end supports should be considered in design of the forms and shoring including sills or other foundation support.

*If supported by tests by a recognized commercial testing laboratory

7.5.3 *Construction accessories*—Hold-down or push-down devices for deflected cables or strands should be provided in the casting bed or forms. All openings, offsets, brackets, and all other items required in the concrete work should be provided for in the formwork. Bearing plates, anchorage assemblies, prestressing steel, conduits, tube enclosures, and lifting devices shown or specified to be set in concrete must be accurately located with formwork templates and anchored to remain within the tolerances given on contract drawings and specifications. Quality and strength of these accessories should be as specified.

7.5.4 *Tolerances*—Suggested tolerances for job site precast and plant manufactured precast prestressed concrete members are given in ACI 117 and the PCI report on tolerances.*

7.5.5 *Special provisions for curing and for safety of workmen*—Where required to allow early reuse of forms, provisions should be made to use such accelerated curing processes as steam curing, vacuum processing, or other approved methods.

Safety shields should be provided at end anchorages of prestressing beds or where necessary for the protection of workmen or equipment against possible breakage of prestressing strands, cables, or other assemblies during prestressing or casting operation.

7.6—Forms for precast concrete construction

7.6.1 *Discussion*—This type of form is used for precast concrete items that may be either load- or nonload-bearing members for structural or architectural uses.

7.6.2 *Construction*—Exterior braces only should be used when exposed metal or filled-in pockets resulting from the use of metal ties would present an objectionable appearance.

To assure uniformity of appearance in the cast members or units, particularly in adjacent units where differences in texture and/or color would be visually apparent, care should be taken that the contact surfaces or forms or form liners are of uniform quality and texture.

Form oil or retardant coatings (nonstaining, if required) should be applied uniformly and in accordance with manufacturers' recommendations for this particular class of work.

7.6.3 *Accessories*—It is particularly important in this class of work that positive and rigid devices be used to insure proper location of reinforcement. All openings, cutouts, offsets, inserts, lift rings, and connection devices required to be set in concrete must be accurately located and securely anchored in the formwork.

The finished surfaces of members should be free of lift rings and other erection items where same will be exposed, will interfere with the proper placing of precast members or other materials, or will be subject to corrosion. Such items should be removed in such a manner that no remaining metal will be subject to corrosion.

Quality and strength of these accessories should be as required by contract drawings and specifications, but the lifting devices or other accessories not called for in contract drawings are the responsibility of the contractor.

7.6.4 *Tolerances*—Suggested tolerances for precast concrete construction are listed in ACI 117.

7.6.5 *Removal of forms*—Precast members or units should be removed from forms only after the concrete has reached a specified strength as determined by the field-cured test cylinders or beams and job history of concrete curing.

Where required to allow early reuse of forms, provisions may be made to use accelerated curing processes such as steam curing, vacuum processing, or other approved methods.

Methods of lifting precast units from forms should be approved by the engineer/architect.

7.7—Use of precast concrete for forms

7.7.1 *Discussion*—Precast concrete panels or molds have been used as forms for cast-in-place and precast concrete, either as permanent, integrated forms or as removable, reusable forms. They have been used for both structural and architectural concrete, designed either as structurally composite with the cast-in-place material or merely to provide a desired quality of outer surface, and in some cases to serve both of these purposes. Concrete form units may be either plain, reinforced, or prestressed, cast in the factory or at the job site. The most common use of precast concrete form units has been for elevated slabs acting compositely with topping concrete.

7.7.2 *Design*

7.7.2.1 *Responsibility for design*—Where the integrated form is to act compositely with the structure concrete, the form panel should be designed by the engineer/architect who should also indicate what additional external support is required for the permanent forms. For permanent forms intended principally to achieve a desired architectural effect, the engineer/architect may specify surface finish and desired minimum thickness of architectural material. Design and layout of temporary forms and supporting systems should normally be the responsibility of the contractor.

7.7.2.2 *Connections*—Connection details should be planned to overcome problems of mating precast members to each other and to the existing or cast-in-place structure.

7.7.2.3 *Bonding concrete form to concrete structure*—Effective bond between precast form unit and the structure concrete is essential, and may be achieved by: (1) special treatment such as grooving or roughening the form face in contact with the structure concrete; (2) use of anchoring devices extending across the interface between form panel and structure concrete; and (3) a combination of (1) and (2). Lifting hooks in a form

*"Tolerances for Precast and Prestressed Concrete," 1985, available from Prestressed Concrete Institute, Chicago.

unit may be designed to serve also as anchors or shear connectors.

7.7.2.4 *Code requirements*—Precast concrete forms used in composite design with cast-in-place concrete should be designed in accordance with "Building Code Requirements for Reinforced Concrete," (ACI 318).

7.7.3 *During and after concreting*

7.7.3.1 *Vibration*—Thorough consolidation of site cast concrete is required to prevent voids which would interrupt the bond of the form to structure concrete, but sufficient care must be exercised to prevent damage of concrete panels by contact with vibrators.

7.7.3.2 *Protection of architectural finish*—Care should be taken to avoid spilling fresh concrete on exposed surfaces, and any spilled or leaked concrete must be thoroughly removed before it has hardened. After concreting, protection of precast architectural concrete form facings may need to be considered.

7.8—Forms for concrete placed under water

7.8.1 *Discussion*—There are two basic approaches to the problem of placing concrete under water. The concrete may be mixed in the conventional manner and then placed by special methods, or the preplaced aggregate method may be used.

In the first approach, placement may be made by either pump, underwater bucket, or the more common tremie. The tremie is a steel pipe, suspended vertically in the water, with a hopper attached to the upper end above the water surface. The lower end of the pipe extends to the bottom of the area to be concreted. This pipe is charged with concrete from the surface, taking care to force any water from the pipe ahead of the concrete. Once the pipe is filled with concrete, it is kept full and its bottom must be kept immersed in the fresh concrete.

In the second approach, the forms are filled with coarse aggregate which is then grouted so that the voids around the aggregate are filled. The grout is introduced at the bottom and the water is displaced upward as the grout rises.

7.8.2 *Underwater bucket and tremie*

7.8.2.1 *Design*—Forms for underwater concreting are designed with the same considerations as other forms covered in Section 2.2 except that the density of the submerged concrete may be reduced by the weight of the water displaced. However, because of large pressures which can develop due to the head developed in the tremie, loads should be evaluated by personnel experienced in this type of work. Some designs have ignored the effects of submergence, because this results in a practical design which is sturdy enough to withstand the extra rigors of underwater conditions.

In tidal zones, forms should be designed for the lowest possible water level. Changes in construction schedules may transform a planned submerged placement to one made above water, thus losing the offsetting water pressure.

7.8.2.2 *Construction*—Underwater forms should be built on the surface in large units insofar as possible, because final positioning and fitting when done underwater by divers is slow and costly. For this reason, foundations should be kept simple in shape, and forms should be free of complex bracing and connection details. Through-ties, which could interfere with the concrete placing should be avoided, insofar as possible.

Forms must be carefully fitted and secured to adjacent materials and/or construction to avoid loss of mortar under pressure developed. If there is any flow past the form, small openings in the form should be avoided as they will permit washing or scouring of the fresh concrete.

When it is intended to permit concrete to overflow the form and screed it off to grade, it is essential that the form is positioned to the proper grade and is detailed so that the overflow will not interfere with the proposed method and devices for stripping.

Forms should be well detailed, and such details should be scrupulously followed so that divers employed to remove the form may visualize and plan their work before descending.

Multiuse forms may have special devices for positioning forms from above water and special stripping devices such as hydraulic jacks which permit releasing the form from the surface.

7.8.3 *Preplaced aggregate*

7.8.3.1 *Design*—The formwork should be designed with the same considerations as mentioned previously in Section 7.2.2, keeping in mind the submerged condition.

7.8.3.2 *Construction*—It is important to assure that silt is excluded from the forms because silt chokes the voids in the aggregate and interferes with the flow of grout. If left adhering to the aggregate, it may reduce the bond between the aggregate and the grout.

The inspection of the forms before concrete placement should verify that the perimeters of the forms are effectively sealed against the leakage of grout or the intrusion of silt or other fines.

CHAPTER 8—REFERENCES

8.1—Recommended references

Documents of the various standards-writing organizations whose procedures are recommended in this guide are listed below with their serial designation, including year of adoption or revision. The documents listed were the current versions at the time this guide was prepared. Since some of the cited recommendations are revised frequently, generally in minor detail only, the user of this guide should check with the sponsoring organization if it is desired to refer to the latest revision.

American Concrete Institute

116R-85 Cement and Concrete Terminology

117-81 — Standard Tolerances for Concrete Construction and Materials

207.1R-70 (Reapproved 1980) — Mass Concrete for Dams and Other Massive Structures

224R-80 (Revised 1984) — Control of Cracking in Concrete Structures

303R-74 (Revised 1982) — Guide to Cast-in-Place Architectural Concrete Practice

304.1R-69 — Preplaced Aggregate Concrete for Structural and Mass Concrete

304.2R-71 (Revised 1982) — Placing Concrete by Pumping Methods

305R-77 (Revised 1982) — Hot Weather Concreting

306R-78 (Revised 1983) — Cold Weather Concreting

309.2R-82 — Identification and Control of Consolidation-Related Surface Defects in Formed Concrete

313.77 (Revised 1983) — Recommended Practice for Design and Construction of Concrete Bins, Silos, and Bunkers for Storing Granular Materials

318-83 — Building Code Requirements for Reinforced Concrete

332R-84 — Guide to Residential Cast-in-Place Concrete Construction

344R-70 (Reapproved 1981) — Design and Construction of Circular Prestressed Concrete Structures

347.1R-69 — Precast Concrete Units Used as Forms for Cast-in-Place Concrete

359-83 — Code for Concrete Reactor Vessels and Containments

American National Standards Institute

A48.1-1985 — Forms for One-Way Concrete Joist Construction

A48.2-1985 — Forms for Two-Way Concrete Joist Construction

A58.1-1982 — Minimum Design Loads for Buildings and Other Structures

A208.1-1979 — Mat-Formed Wood Particle Board

American Plywood Association

Plywood Design Specification, 1985

ASTM

A 446-76 (Reapproved 1981) — Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality

C 532-66 (Reapproved 1979) — Standard Specification for Structural Insulating Formboard (Cellulosic Fiber)

Canadian Standards Association

CAN3-086-M80 — Code for Engineering Design in Wood

National Forest Products Association

National Design Specification for Wood Construction, 1982

U.S. Department of Commerce

LLB-810a — Hardboard Concrete Form Liners (Simplified Practice Recommendation)

PS1-83 — Construction and Industrial Plywood

PS20-70 — American Softwood Lumber

These publications may be obtained from the following organizations:

American Concrete Institute
P.O. Box 19150
Detroit, MI 48219-0150

American National Standards Institute
1430 Broadway
New York, NY 10018

American Plywood Association
P.O. Box 11700
Tacoma, WA 98411

ASTM
1916 Race St.
Philadelphia, PA 19103

Canadian Standards Association
178 Rexdale Blvd.
Rexdale, Ontario M9W 1R3
Canada

National Forest Products Association
1250 Connecticut Ave., NW
Washington, DC 20036

U.S. Department of Commerce
publications available from:
U.S. Government Printing Office
Washington, DC 20402

8.2—Cited references

Additional information sources on which the committee recommendations are based are listed in Sections 1.5, 2.9, and 4.5

This report was submitted to letter ballot of the committee and was approved in accordance with ACI balloting procedures.

APPENDIX—METRIC AND SI EQUIVALENTS

Conversion factors

To convert from	to	multiply by
inches (in.)	centimeters (cm)	2.54
inches (in.)	millimeters (mm)	25.4
feet (ft)	meters (m)	0.3048
miles	kilometers (km)	1.609
pounds-force (lb)	kilograms-force (kgf)	0.4536
kilograms-force	newtons (N)	9.807
pounds-force (lb)	newtons (N)	4.448
pounds-force per lineal foot (lb/ft)	kilograms-force per meter (kgf/m)	1.488
pounds-force per lineal foot (lb/ft)	newtons per meter (N/m)	14.59
pounds-force per square inch (lb/in.²)	kilograms-force per square centimeter (kgf/cm²)	0.0703
pounds-force per square inch (lb/in.²)	kilopascals* (kPa)	6.895
pounds-force per square foot (lb/ft²)	kilograms-force per square meter (kgf/m²)	4.882
pounds-force per square foot (lb/ft²)	kilograms-force square centimeter (kgf/cm²)	0.0004882
pounds-force per square foot (lb/ft²)	kilopascals (kPa)	0.04788
pounds-mass per cubic foot (lb/ft³)	kilograms per cubic meter (kg/m³)	16.02
temperature, Fahrenheit (F)	temperature, Celsius (C)	use $t_c = (t_f - 32)/1.8$

*One newton per square meter is a pascal.

Conversions of nonhomogeneous equations

Section 2.2.2 — *Lateral pressure of concrete (metric equivalents)*

$$p_u = 0.24h_u \qquad (2\text{-}1)$$

a. For columns

$$p_u = 0.073 + \frac{8.0R_u}{T_c + 17.8} \qquad (2\text{-}2)$$

(maximum of 1.47 kgf/cm² or $0.24h_u$, whichever is least)

b. For walls, rate of placement not exceeding 2 m/hr

$$p_u = 0.073 + \frac{8.0R_u}{T_c + 17.8} \qquad (2\text{-}2a)$$

(maximum of 0.98 kgf/cm² or $0.24h_u$, whichever is least)

c. For walls, rate of placement 2 to 3 m/hr

$$p_u = 0.073 + \frac{11.78}{T_c + 17.8} + \frac{2.49R_u}{T_c + 17.8} \qquad (2\text{-}3)$$

(maximum 0.98 kgf/cm² or $0.24h_u$, whichever is least)

where

p_u = lateral pressure, kgf/cm²
R_u = rate of placement, m/hr
T_c = temperature of concrete in the forms, deg C
h_u = height of fresh concrete above point considered, m

Section 2.2.2—*Lateral pressure of concrete (SI equivalents)*

$$p_u = 23.5h_u \qquad (2\text{-}1)$$

For columns

$$p_u = 7.2 + \frac{785R_u}{T_c + 17.8} \qquad (2\text{-}2)$$

(maximum of 144 kPa or $23.5h_u$, whichever is least)

For walls, rate of placement not exceeding 2 m/hr

$$p_u = 7.2 + \frac{785R_u}{T_c + 17.8} \qquad (2\text{-}2a)$$

(maximum of 95.8 kPa or $23.5h_u$, whichever is least)

For walls, rate of placement from 2 to 3 m/hr

$$p_u = 7.2 + \frac{1156}{T_c + 17.8} + \frac{244R_u}{T_c + 17.8} \qquad (2\text{-}3)$$

(maximum 95.8 kPa or $23.5h_u$, whichever is least)

where

p_u = lateral pressure, kPa
R_u = rate of placement, m/hr
T_c = temperature of concrete in the forms, deg C
h_u = height of fresh concrete above point considered, m

Section 7.3.2.4—*Lateral pressure of concrete (metric equivalents)*

The lateral pressure of fresh concrete to be used in designing forms, ties, bracing, and wales may be calculated as follows:

$$p_u = c_1 + \frac{5.35R_u}{T_c + 17.8}$$

where

c_1 = 0.05
p_u = lateral pressure, kgf/cm²
R_u = rate of placement, m/hr
T_c = temperature of concrete in the forms, deg C

Section 7.3.2.4—*Lateral pressure of concrete (SI equivalents)*

The lateral pressure of fresh concrete to be used in designing forms, ties, bracing, and wales may be calculated as follows:

$$p_u = c_1 + \frac{524R_u}{T_c + 17.8}$$

where

c_1 = 4.79
p_u = lateral pressure, kPa
R_u = rate of placement, m/hr
T_c = temperature of concrete in the forms, deg C

Batching, Mixing, and Job Control of Lightweight Concrete

Reported by ACI Committee 304

This report covers many of the practical aspects of batching of lightweight aggregate concrete and includes comments on mixing and job controls. Procedures for batching are covered in detail, enabling the user to achieve proper yield under varying conditions of moisture and unit weight of aggregates. Absorbed water and free water are explained. Pertinent details of mixer operation and job controls are also covered to assure a quality product meeting applicable job specifications.

Keywords: absorption; aggregates; air entrainment; **batching; bulk density;** coarse aggregates; **density (mass/volume);** fine aggregates; **lightweight aggregate concretes; lightweight aggregates;** mixers; **mixing;** mix proportioning; moisture content; quality control; saturation; **slump tests;** voids; water; **weight measurement;** wetting.

1 — INTRODUCTION

Measuring, mixing, transporting, and placing operations for lightweight concrete are similar to comparable procedures for normal weight concrete. However, there are certain differences, especially in proportioning and batching procedures, that should be considered in order to produce a finished product of the highest quality. The weight and absorptive properties of lightweight aggregates are different and should be properly considered. Every effort has been made to coordinate these batching methods with the basic principles set forth in ACI 211.2. Other batching

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methods currently being used in various locations may also be employed. This report also describes batching methods developed over 15 years ago in Southern California and used in several other locations for the coarse lightweight aggregates to correct for changes in weight and moisture content to insure proper yield. It also covers batching of lightweight fine aggregates using a modification of the method used for coarse lightweight aggregates.

Quality control for plastic lightweight concrete requires special emphasis with regard to yield, aggregate measuring, and batching methods along with the control of water for slump and for aggregate absorption.

2 — MEASURING AND BATCHING

2.1 — Free Water and Absorbed Water

One of the first considerations in batching lightweight concrete mixtures is a proper understanding of the water used in the mixture. The total water used per unit volume is divided into two components or parts. One part is the water absorbed by the aggregates while the other is similar to that in normal weight aggregate concrete and is classified as free water. Free water controls the slump and, when mixed with a given quantity of cement, establishes the strength of the paste. The amount of absorbed water will vary with different lightweight materials, presoaking, and mixing times. Absorbed water does not change the volume of the aggregates or concrete because it is inside the aggregate. Most importantly, absorbed water does not affect the water-cement ratio or the slump of the concrete.

2.2 — Absolute Volumes

Lightweight concrete uses lightweight aggregate particles in place of normal weight aggregates to the extent necessary to achieve the total weight desired in the hardened concrete. The space that the aggregates occupy within the concrete is called their absolute volume. The sum of the absolute volumes of all the ingredients including air must equal the required volume of mixed concrete.

By definition, the absolute volume of a loose granular material is the net volume of solid material after removing the voids or air spaces between the particles. The absolute volume may be calculated by either of the following formulas:

$$\text{Abs. Vol. in cu ft} = \frac{\text{Weight of loose material in lb}}{\text{Specific gravity of material} \times 62.4}$$

$$\text{Abs. Vol. in m}^3 = \frac{\text{Weight of loose material in kg}}{\text{Specific gravity of material} \times 1000}$$

2.2.1 – Bulk Specific Gravity (Specific Gravity Factor, Dry) of Coarse and Fine Aggregate

The methods used to determine the bulk specific gravity of normal weight aggregates cannot be used with lightweight aggregates because of their variable absorption rates and the resulting difficulty of determining their displaced volume in water. Methods described in Appendix A and B of ACI 211.2 for measuring the specific gravity factor (dry) and the moisture content give reliable results.

For coarse lightweight aggregate, this method consists essentially of immersing a suitably sized sample (about 1000-1500 g) for 24 ± 4 hr in water, allowing it to surface dry in air or spin drying it in a centrifuge, and then measuring its apparent specific gravity in this saturated surface dry (SSD) condition with either a pycnometer or by the displacement method described in ASTM C 127. Half of the SSD sample is oven dried to determine its percentage of absorption. The SSD specific gravity is then reduced by the percentage of absorption to obtain the oven dry bulk specific gravity or the specific gravity factor (dry). For example, if the SSD specific gravity is 1.41 and the absorption is 13.6 percent, the oven dry bulk specific gravity is:

$$\frac{1.41}{1.0 + 0.136} = \frac{1.41}{1.136} = 1.24$$

For lightweight fine aggregate, the oven dry bulk specific gravity is determined in much the same manner as for the coarse lightweight material. However, it is difficult to visually determine the SSD condition and the spin dry procedure (Reference 12) or the ASTM C 128 may give more satisfactory results. Another procedure for determining the bulk specific gravity using all dry materials, which employs a flow cone sand testing apparatus, is described in Reference 10.

2.2.2 – Unit Weight Variations

The unit weight of lightweight aggregate varies depending on the raw materials used and the size of the aggregate. Smaller particles usually have higher densities, specific gravities, and unit weights than larger particles. Unit weights also vary due to changes in absorption or moisture content. If the lightweight aggregates are batched without adjusting for these variations in unit weight, problems of over or under yield of the concrete can result. To prevent such problems, various field adjustments are suggested in the standard on Proportioning Lightweight Concrete, ACI 211.2. Essentially these field adjustments consists of changing the batch weights of the lightweight aggregates, both coarse and fine, to insure that the resulting concrete produces the intended volume or yield.

The dry loose unit weight of aggregate depends on its specific gravity, on the grading, and on the shape and size of the particles. Angular shaped crushed aggregates have more voids or unfilled spaces between the aggregate particles than rounded or spherically shaped pieces. Poorly graded aggregate (i.e., all one

TABLE 1 — Lightweight concrete laboratory mix proportion

Item	Batch wt.	Loose vol.	Absol. vol.
	Quantities per cubic yard		
Cement	564 lb	6.0 sks.	2.86 cf
Free water	305	36.6 gal.	4.89
Entr'd air by AEA	per Mfg.	6%	1.62
Coarse ltwt. (dry)	774	17.0 cf	8.86
Fine ltwt. (dry)	952	15.9 cf	8.77
Absorb. water max.	224	26.9 gal.	—
TOTALS	2821		27.00
Wet plastic unit weight of concrete = 2821/27.00 = 104.5 pcf			
	Quantities per cubic meter		
Cement	335 kg	0.222 m³	0.106 m³
Free water	181	0.181	0.181
Entr'd air by AEA	per Mfg.	6%	0.060
Coarse ltwt. (dry)	459	0.630	0.328
Fine ltwt. (dry)	565	0.590	0.325
Absorb. water max.	134	0.134	—
TOTALS	1674		1.000
Wet plastic unit weight of concrete = 1674/1.000 = 1674 kg/m³			

size) generally has more voids than a uniformly graded material which has enough smaller pieces to fit into the voids between the larger particles.

Numerous routine tests of both natural and lightweight aggregates show an amazingly close correlation of the void content for specific products being produced by a given plant over a long period. If changes are made in the source of raw materials, in crushing or screening equipment, or in production methods, this could result in a different void content. With no such major changes, the variation in the void content will generally result in less than 1.0 percent change in yield of the mixture. Different sized materials from the same production facility may have a different but also a relatively constant void content. Each production facility has its own characteristic void content value for each size aggregate being produced, and this information can usually be obtained from the source.

The absolute volume of the specific lightweight materials in a given container would be the volume of material remaining after the volume of voids has been subtracted from it. In other words, if the unfilled void space was 44 percent or 0.44, then the absolute volume would be 1.00 − 0.44 = 0.56 or 56 percent. Every loose unit volume of lightweight aggregate in this case will add only 56 percent of that volume as net solids or absolute volume to the total volume of the concrete. The absolute volume, or the displaced volume in the concrete, for a given lightweight material will remain the same even though its density changes or its moisture absorption changes.

The proper usage of these basic principles makes it possible for any ready-mixed concrete producer to batch and deliver lightweight concrete at the proper slump and yield for any job.

2.3 — Batching Coarse Aggregate

2.3.1 — Mix Proportions

For illustration purposes, a typical lightweight concrete mixture prepared in a laboratory is shown in Table 1. This mixture was proportioned by the weight method described in ACI 211.2. The quantities per cubic yard and per cubic meter of concrete are shown separately. The specification requirements for the lightweight concrete and the properties of the lightweight coarse and fine aggregate are given as follows:

Specifications: 3000 psi (20.7 MPa) at 28 days, slump 3-4 in. (75-100 mm), air entrainment 6 ± 1 percent, air dry weight max. 100 pcf (1602 kg/m³), wet plastic weight max. 105 pcf (1682 kg/m³), max. aggregate ¾ in. (19 mm).

Aggregate Properties on Laboratory Samples: Ltwt. coarse: Gradation meets ASTM C 330, oven dry loose weight = 45.5 pcf (730 kg/m³), Sp. Grav. factor (dry) 1.40, 24 hr absorption 12.6 percent. Ltwt. fines: Gradation meets ASTM C 330, oven dry loose weight = 59.7 pcf (956 kg/m³), Sp. Grav. factor (dry) 1.74, 24 hr absorption 13.4 percent.

The quantity of lightweight aggregate is shown in Table 1 on an oven dry basis with the absorbed water shown as a separate item. In this example, the batch weights (based on the given dry loose unit weight) are tabulated and the loose volume of the dry coarse and fine aggregates is shown. The absolute volume is calculated from these batch weights using the oven dry specific gravity factor.

To obtain proper yield of concrete, it is necessary to maintain the same absolute volumes of lightweight aggregates in each batch of concrete by adjusting the batch weights to compensate for changes in unit weights. This may be done by making standard unit weight tests on the lightweight aggregates frequently during batching operations and adjusting the batch weights to reflect any changes that may occur in these unit weights. Although this practice is followed successfully in many areas of the country, it may be rather time-consuming in a busy production facility. The alternate batching system described in this report has been developed as a faster method. Either method produces satisfactory results. The principal difference in the two systems is that the latter method uses a much larger container for measuring the unit weight — the weighing hopper. In addition, it provides automatic yield adjustments for every single batch of lightweight concrete.

2.3.2 — Calibrating the Weighing Hopper

The system can be set up for virtually any batching facility that employs a hopper or bin for weighing materials. The first operation is to determine the volume of this weighing hopper.

When the discharge gate in the overhead bin containing the lightweight coarse aggregate is opened, the material will flow into the weighing hopper until it builds up to the level of the discharge gate. Some plants may be slightly different than others but suitable modifications, as shown in Fig. 1, can be made in the overhead bins, in the weighing hopper, or both to allow the weighing hopper to be filled to a prescribed level each time.

The volume of lightweight aggregate in this filled weighing hopper can be calibrated for most batching plants in the following manner. The total weight of the material (either dry or containing absorbed water) in the filled hopper can be read directly from the weight scales. The hopper is then discharged into a dump truck and the unit weight of three or four samples of loose material is determined in a suitable container. The total hopper weight divided by the average unit weight will give the total volume of the material in the weighing hopper in cubic feet or in cubic meters. As an example, if the net weight of the filled hopper is 4650 lb (2110 kg) and the average unit weight of the material in it is 48.2 pcf (772 kg/m³), the volume is simply 4650/48.2 = 96.5 cf, or, 2110/772 = 2.73 m³. This calibration procedure should be performed three times to insure valid measurements. A new calibration might be necessary if the source of lightweight aggregate is changed, since the angle of repose could vary, which would change the overall volume in the weighing hopper. If no major changes occur in the lightweight aggregates, then one calibration will suffice for several months.

2.3.3 — Batching Chart

For the purposes of illustration, assume that the calibrated volume of a given weighing hopper was

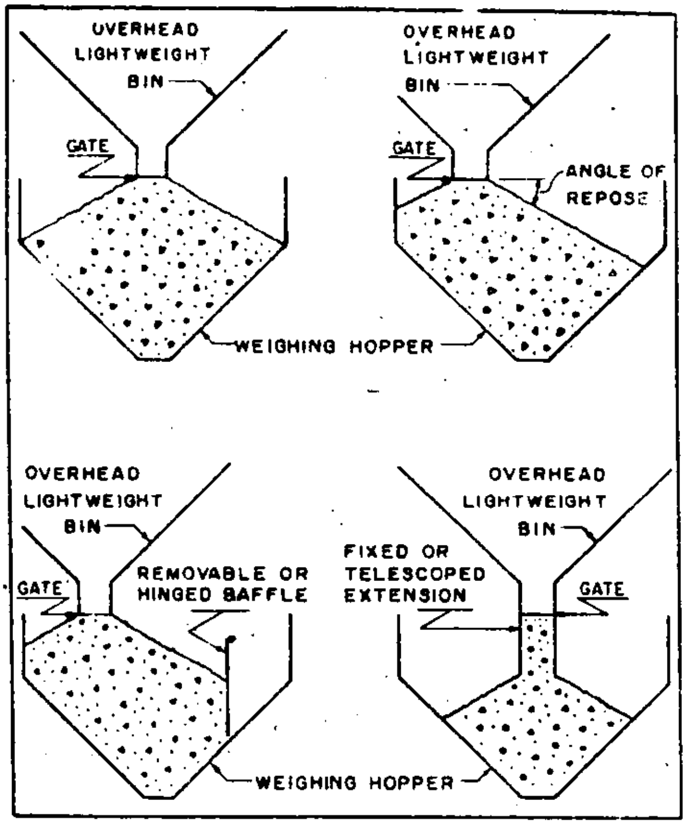

Fig. 1 — Overhead bin and weighing hopper arrangements.

found as shown to be 96.5 cf (2.73 m³) and that each truck mixer is to be loaded with 7.0 yd³ or with 5.0 m³ of the lightweight mixture shown in Table 1. In this case the total loose volume of lightweight coarse would be 7.0 × 17.0 = 119 cf or 5.0 × 0.63 = 3.15 m³. A simple chart is prepared for the batch plant operator such as Table 2 to mix 7.0 yd³ or Table 3 to mix 5.0 m³.

To prepare this chart, the possible range of full hopper weights is listed in the first or left hand column. Since the loose volume in the full hopper is 96.5 cf (2.73 m³), the loose unit weight per cubic foot or per cubic meter (either damp or dry) may be calculated by taking the weight in the first column and dividing this by 96.5 cf (2.73 m³). These values are shown in the second column of Table 2 or Table 3. The remaining volume of loose material needed to complete the 7.0 yd³ batch is simply 119 minus 96.5 or 22.5 cf in Table 2, or 3.15 minus 2.73 or 0.42 m³ in Table 3.

To batch the concrete, the weighing hopper is first filled with lightweight coarse aggregate, and its weight is determined on the scales. The line of the chart on which the weight in the first column is closest to this scale weight is noted and the contents of the weighing hopper are discharged. The additional volume of 22.5 cf or 0.42 m³ is added to the hopper based upon the calculated weights shown in the third column on the same line of Table 2 or 3. The calculated weights shown in the third column are obtained

TABLE 2 — Batching chart for 7.0 cu yd of concrete

Full weighing hopper volume = 96.5 cf

(1) Weight of first hopper filled lb	(2) Loose unit wt. pcf	(3) Weight in second hopper lb	(4) Total wt. of aggreg. lb	(5) lb of Aggregate for 1.0 cu yd
4000	41.4	932	4932	704
4050	42.0	945	4995	714
4100	42.5	956	5056	723
4150	43.0	968	5118	731
4200	43.5	979	5179	740
[illegible]	44.0	990	5240	748
[illegible]	44.6	1004	5304	758
[illegible]	45.1	1015	5365	767
4400	45.6	1026	5426	775
4450	46.1	1037	5487	784
4500	46.6	1049	5549	792
4550	47.2	1062	5612	802
4600	47.7	1073	5673	811
4650	48.2	1085	5735	819
4700	48.7	1096	5796	828
4750	49.2	1107	5857	836
4800	49.7	1118	5918	845
4850	50.3	1132	5982	855
4900	50.8	1143	6043	864

TABLE 3 — Batching chart for 5.0 m³ of concrete

Full weighing hopper volume = 2.73 m³

(1) Weight of first hopper filled kg	(2) Loose unit wt. kg/m³	(3) Weight in second hopper kg	(4) Total wt. of aggreg. kg	(5) kg of aggregate for 1.0 m³
1800	659	277	2077	415
1825	668	281	2106	421
1850	678	285	2135	427
1875	687	289	2164	433
1900	696	292	2192	438
1925	705	296	2221	444
1950	714	300	2250	450
1975	723	304	2279	455
2000	733	308	2308	462
2025	742	312	2337	467
2050	751	315	2365	473
2075	760	319	2394	479
2100	769	323	2423	484
2125	778	327	2452	490
2150	788	331	2481	496
2175	797	335	2510	502
2200	806	339	2539	508
2225	815	342	2567	513
2250	824	346	2596	519

by multiplying the unit weight shown in the second column by the required volume of 22.5 cf or 0.42 m³.

Other tables similar to Table 2 or 3 can be prepared in advance for any mix proportion assuming the basic full hopper volume will remain the same. The batch plant operator simply notes the scale weight of the first full hopper and from this table can immediately determine the weight needed to complete the batch. This same table can be programmed into an automatic electronically controlled batching facility or it could be used in a semi-automatic plant where all of the ingredients except the lightweight aggregates are batched electronically.

If it is desired to record the total weight of coarse lightweight aggregate on the delivery ticket for any given truck, the total weights as batched are shown in the fourth column of either Table 2 or Table 3. Also if the unit weight of the aggregate is required on the delivery ticket, the value shown in the second column provides this information.

If batches less than a full truckload might be needed, these could be batched in one cubic yard (or one cubic meter) increments using the unit weight of aggregate determined on the immediately preceding batch multiplied by the loose volume shown on the mix proportion. These batch weights are shown in the fifth column of Table 2 or Table 3.

2.4 — Batching Lightweight Fine Aggregate

It is not practical to batch the lightweight fine aggregate by this same method since its volume changes due to variable bulking with different amounts of surface water. For this reason, the lightweight fines are batched by weight in much the same manner as natural sand with allowances made for total moisture content.

Since the moisture in lightweight fines may be partly absorbed water as well as surface or free water, the moisture meters used in batch plant storage bins for natural sand have not been satisfactory for lightweight sand. Satisfactory batching results have been obtained by drying a small sample (about 500 g) of the lightweight sand being used in a suitable container to a constant weight at a temperature of 212 to 230 F (100 to 110 C). The total moisture (absorbed plus surface moisture) is calculated by comparing the moist weight of the sample to its dry weight. Moisture tests should be conducted at least once per day or whenever a fresh supply of lightweight sand is introduced which has a different moisture content.

To adjust for the proper amount of lightweight fines, the oven dry unit weight of the material being used is determined as indicated above. If this dry unit weight differs from that shown on the laboratory mix proportion [59.7 pcf (956 kg/m³) shown in the example] then the dry batch weight is changed by multiplying the loose volume [15.9 cf (0.590 m³)] by the new dry unit weight just determined. This dry batch weight is increased by the moisture content as previously determined to give the actual scale weight to be used.

3 — MIXING

The absorptive properties of lightweight aggregates should be given consideration during mixing. The time rate of absorption as well as the maximum total absorption must be properly integrated into the mixing cycle in order to control the slump accurately.

3.1 — Charging Mixers

The sequence of introducing the ingredients for lightweight concrete into a mixer may vary from one plant to another. Once acceptable procedures for both wetting and batching have been established, it is important to repeat these as closely as possible at all times to assume uniformity. Weather conditions such as ambient temperature and humidity can exert significant influences on any concrete production and should be properly considered.

3.1.1 – Plant Mixers

Stationary plant mixers are commonly used in precasting or prestressing operations and occassionally on building sites where concrete is not moved a great distance. They may also be used at a ready-mixed production plant for complete premixing or for partial premixing (shrink mixing) with the concrete later being fully mixed and transported to the jobsite in mixer trucks.

Dry or moist lightweight aggregates should be placed in the mixer first, followed by the required water, cement, and any specified additives. Lightweight fines should be added after the coarse aggregates when lightweight sand is being used in the concrete.

After all of the ingredients have been fed into the plant mixer, it should be operated at mixing speed produce a complete mix that will meet the evaluation tests as described in ASTM C 94. When stationary mixers are used for the purpose of partial or shrink mixing, they are only required to blend the materials together since mixing is completed in the truck mixer. If the lightweight aggregate has not reached its full saturation, further aborption during and after mixing may cause the mix to stiffen.

3.1.2 – Truck Mixers

Charging or loading a truck mixer follows the same general practice used in stationary mixers. Larger volumes of lightweight concrete can sometimes be hauled in truck mixers without exceeding the legal weight or axle load limits. However, the volume of concrete in the drum should not exceed 63 percent of the drum volume when used as a mixer nor 80 percent of this volume when used as an agitator in accordance with ASTM C 94.

3.2 — Mixer Operation

Since most concrete, both normal and lightweight, is handled in truck mixers, it is important to understand some aspects of truck operation. Delivery time and weather effects have an important role in slump control. These variables may require changes in the amount of water needed to produce the desired slump.

3.2.1 – Transportation and Waiting Time

Construction jobs at different distances from the batch plant require longer or shorter haul periods, and it is not uncommon to have a delay in unloading. Th factors make it difficult to determine the total e that a mixture will be in the drum for any particular load. Some lightweight aggregates may continue to absorb water with time even though prewetted. Prewetting slows the rate of absorption but does not necessarily eliminate absorption. Some operators hold back 2 to 3 gal. of water per cu yd (10 to 15 liters per m^3) to make certain that the batch is not too wet upon arrival. It is often necessary and entirely permissible to add water to a lightweight concrete mix on the job to replace free water which has been absorbed by the lightweight aggregate in order to bring the concrete back up to the desired slump.

Truck mixers should be operated at prescribed mixing speeds for the range of total revolutions required to produce complete mixing, normally 70 to 100 revolutions, and then be slowed to agitating speed. Just prior to unloading, it is suggested that the mixer be rotated at mixing speed for one or two min. It is also desirable to stop the unloading operation when th drum is about half empty and to reverse the drum in the mixing direction for three or four revolutions at mixing speed to assure continued uniformity of the mixed material being delivered.

3.2.2 – Temperature Effects

The temperature of the individual ingredients and the resulting temperature of the concrete mixture affect total water requirements. Temperatures from 50 to 85 F (10 to 30 C) generally have no adverse effects on the mix. Higher temperatures generally increase mixing water requirements. During hot weather construction, prewetting of the coarse lightweight aggregate will help to reduce the temperature of the concrete and will also reduce the amount of water absorbed from the mix by this material. Premature stiffening or loss of slump may be caused by high mix temperature and relative humidity and have nothing to do with a shortage of water in the mix. Water added under these conditions could produce serious losses in strength and other properties.

3.2.3 – Adding Water at the Jobsite

Water to replace that lost through absorption may be added to the mix at the jobsite to produce the specified slump without endangering the strength and other properties of the mix and without changing the volume of the concrete. Approximately 1 gal. or about 10 lb of water per cu yd (5 to 6 liters per m^3) will increase the slump by 1 in. (25 mm). When water is added, the mixer should be operated at mixing speed for a minimum of 30 revolutions before it is discharged.

4 — JOB CONTROLS

Control tests discussed here pertain primarily to the mixed lightweight concrete. However, there are other tests which can be made on the individual ingredients, particularly on the lightweight aggregates. The latter tests are covered in ASTM C 330, "Specifications for Lightweight Aggregate for Structural Concrete."

Samples of concrete for field or jobsite tests should always be taken from the midportion of the load as it is delivered. Individual samples taken after discharge of approximately 15 percent and before 85 percent of the load has been discharged will satisfy this requirement, following ASTM C 172. All testing methods should be performed in accordance with current ASTM test methods.

4.1 — Slump

The slump test for lightweight concrete is performed exactly the same as for normal weight concrete. The slump of lightweight concrete should be about two-thirds that of normal weight concrete to produce equal workability. This is because the lightweight aggregates weigh less and this reduces the effect of gravity.

The slump of concrete between 50 to 85 F (10 to 30 C) is controlled by the free water in the mix and is independent of the absorbed water. If the specified slump is obtained at the time and point of placement, it can be assumed that the strength and other properties of the mix, as originally designed, have been maintained. Within these stated mix temperatures, additional water may be added upon arrival at the jobsite only if needed to produce the specified slump as delivered in accordance with ASTM C 94. Where the concrete is transported some distance from the truck, particularly if pump placement is used, it is advisable to have comparative slump tests made at the point of placement. In this case, it is important to mention that such samples should be remixed in accordance with ASTM C 172 before conducting the slump tests as described in ASTM C 143.

4.2 — Unit Weight

The unit weight of the plastic concrete is important in the control of lightweight mixtures and in verifying compliance with structural design criteria. In most cases, the job specifications place an upper limit on the air-dry unit weight in accordance with ACI 301 and with ASTM C 567. Since the air-dry weight cannot be measured at the time of placement, the plastic unit weight should be used as a field control.

The unit weight of fresh concrete is determined in a suitably sized and calibrated container according to ASTM C 138. Cylinder cans or molds should not be used as unit weight containers since their volumes are not always equal. If the measured unit weight in the field does not agree within 2 lb per cu ft (30 kg per m^3) above or below the original mix design weight (including the absorbed water in the aggregates), corrective action should be taken. The various corrective measures are described in Section 4.4 on Yield Adjustments.

In addition to the unit weight of the plastic concrete, it is also advisable to monitor the unit weight of the oven dry lightweight aggregates at the batch plant. The current ASTM C 330 provides that these aggregates shall not differ more than 10 percent from the weight used in the mix proportion. A change in dry unit weight of the aggregates of 10 percent on the coarse fraction only would produce a variation of 2 to 3 lb per cu ft (30 to 50 kg per m^3) in the plastic unit weight of the concrete.

If lightweight concrete is to be pumped, the moisture content should be checked to make certain that sufficient saturation has been achieved to avoid excessive absorption as a result of pumping pressure applied to the concrete.

4.3 — Air Content

In conjunction with lightweight concrete, entrained air is frequently used, and its control on the job is an important consideration in the final quality of the concrete. In addition to providing increased resistance to freezing and thawing, air entrainment helps to reduce the weight of these mixes. More importantly, air entrainment produces a more cohesive mix which improves workability and minimizes segregation of the heavier mortar from the lighter aggregate particles.

ASTM C 173, "Measuring the Air Content of Freshly Mixed Concrete by the Volumetric Method" is preferred to ASTM C 231, "Measuring the Air Content of Freshly Mixed Concrete by the Pressure Method." If the pressure method is used, it will measure some of the air within the pores of the lightweight aggregate in addition to the air in the mortar. The usually accepted tolerances on air content also apply to lighweight concrete. However, variations in air content also produce variations in plastic unit weight. Air contents excessively above those specified, can produce substantial reductions in strength, especially in the richer high-strength mixes. An increase in air content of 2 percent can cause a reduction in unit weight in excess of 2 lb per cu ft (30 kg per m^3). This increase in air content should produce only a relatively small strength reduction in lean mixes using a cement content of less than 500 lb per cu yd (300 kg per m^3) but could result in 10 percent strength reduction for richer mixes using 800 lb per cu yd (500 kg per m^3) or more of cement. Therefore, it is imperative to maintain tight controls on air content.

4.4 — Yield Adjustments

Field control of the yield of lightweight concrete is most important. Overyield produces a larger volume of concrete than intended while underyield produces less. Overyield is nearly always associated with a loss in strngth due to a reduction in the net cement content. Underyield results in less concrete being delivered than was expected or ordered.

The unit weight of the plastic concrete is used to measure the yield of a mixture. The weight of all the ingredients that are placed in a mixer drum as given on the delivery ticket is added, or, the entire truck may be weighed before and after discharging. The total weight includes all of the cement, the aggregates, whether wet or dry, and all of the water added. The fresh plastic unit weight divided into the weight of all the in[illegible]nts will give the total volume of concrete in the mixer drum. When the calculated volume is more than 2 percent above or below the volume shown on the delivery ticket, an adjustment is required.

If the change in yield is due to entrained air content, then an adjustment in the amount of air-entraining agent may correct this condition.

If the unit weight measured in the field is greater than the unit wet weight shown on the mix proportion (see Table 1), this would indicate an underyield, conversely if the weight is less, an overyield may occur. When there have been no appreciable changes in the weights of the original lightweight aggregates themselves, in all probability the differences in yield can be attributed to an incorrect amount or an incorrect absolute volume of lightweight aggregates. In this case, steps should be taken at the batch plant to correct the absolute volume of lightweight aggregates used in the concrete as it is being batched.

4.5 — Test Cylinders

Making, storing, and testing concrete cylinders is extremely important on every job. Standard methods should be carefully observed. Failure to follow these standardized procedures may lead to lower test values which may not reflect the true strength of the concrete. Emphasis should be placed on this most important facet of concrete job controls to avoid subsequent disputes or delays.

References

I. Standards and ACI Documents prescribed or mentioned in this Report

The standards of the various standard-producing organizations and ACI documents referred to in this document are listed below with their serial designation, including year of adoption or revision. The standards and reports listed were the latest effort at the time this document was revised. Since some of these publications are revised frequently, generally in minor details only, the user of this document should check directly with the sponsoring group if it is desired to refer to the latest revision.

ACI 211.2-81	Standard Practice for Selecting Proportions for Structural Lightweight Concrete
ACI 301-72 (Revised 81)	Specifications for Structural Concrete for Buildings
ASTM C 94-81	Standard Specification for Ready Mixed Concrete
ASTM C 127-80	Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate
ASTM C 128-79	Standard Test Method for Specific Gravity and Absorption of Fine Aggregate
ASTM C 138-77	Standard Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete
ASTM C 143-78	Standard Test Method for Slump of Portland Cement Concrete
ASTM C 172-71 (1977)	Standard Method of Sampling Fresh Concrete
ASTM C 173-78	Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method
ASTM C 231-81	Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method
ASTM C 330-80	Standard Specification for Lightweight Aggregates for Structural Concrete
ASTM C 567-80	Standard Test Method for Unit Weight of Structural Lightweight Concrete

II. Other Standards and ACI Documents

ACI 213R-79	Guide for Structural Lightweight Aggregate Concrete
ACI 304-73 (Reaffirmed 78)	Recommended Practice for Measuring, Mixing, Transporting and Placing Concrete
ACI 304.2R-71	Placing Concrete by Pumping Methods
ACI 305R-77	Hot Weather Concreting
ACI 306R-78	Cold Weather Concreting
ASTM C 33-81	Standard Specification for Concrete Aggregates

ACI Standards and Committee Reports are generally published in the ACI Manual of Concrete Practice. ACI publications are available from the American Concrete Institute, P.O. Box 19150, Detroit, Michigan, 48219. ASTM standards may be obtained from the American Society for Testing and Materials, 1916 Race Street, Philadelphia, PA, 19103.

III. Additional References

1. "Workability is Easy," *Information Sheet* No. 1, Expanded Shale Clay and Slate Institute, Bethesda, Revised 1965, 3 pp.

2. "Suggested Mix Design for Job Mixed Structural Lightweight Concrete," *Information Sheet* No. 3, Expanded Shale Clay and Slate Institute, Bethesda, Revised 1965, 2 pp.

3. *Design and Control of Concrete Mixtures*, 12th Edition, Portland Cement Association, Skokie, 1979, 140 pp.

4. "Bulking of Sand Due to Moisture," *Concrete Information Sheet* No. ST20, Portland Cement Association, Skokie, 1944, 2 pp.

5. Rielly, William E., "Hydrothermal and Vacuum Saturated Lightweight Aggregate for Pumped Structural Concrete," ACI JOURNAL, *Proceedings* V. 69, No. 7, July 1972, pp. 428-432.

6. Shideler, J. J., "Lightweight-Aggregate Concrete fo Structural Use," ACI JOURNAL, *Proceedings* V. 54, No. 4, Oct. 1957, pp. 299-328.

7. Tobin, Robert E., "Lightweight Ready Mix — A New Approach," *Concrete Products*, V. 70, No. 10, Oct. 1967, 5 pp. Also, *Technical Information Letter* No. 249, National Ready Mixed Concrete Association.

8. Tobin, Robert E., "Handling Lightweight Concrete on the Job," *Lightweight Concrete*, SP-29, American Concrete Institute, Detroit, 1971, pp. 63-71.

9. Tobin, Robert E., "Hydraulic Theory of Concrete Pumping," ACI JOURNAL, *Proceedings* V. 69, No. 8, Aug. 1972, pp. 505-510.

10. Tobin, Robert E., "Flow Cone Sand Tests," ACI JOURNAL, *Proceedings* V. 75, No. 1, Jan. 1978, pp. 1-12.

11. Wills, Milton H., Jr., "Lightweight Aggregate Particle Shape Effect on Structural Concrete," ACI JOURNAL, *Proceedings* V. 71, No. 3, Mar. 1974, pp. 134-142.

This report was submitted to letter ballot of the committee which consists of 27 eligible members: 20 were affirmative, 1 abstention, and 6 were not returned. It has been processed in accordance with the Institute procedure and is approved for publication and discussion.

ACI Committee 304
Measuring, Mixing, Transporting, and Placing Concrete

James L. Cope,* Chairman

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insert this sheet into binder immediately after the tab divider labeled "ASTM Standards"

NOTE ON ASTM STANDARDS

The ASTM Standards which follow are reprinted, with permission, from the *1988 Annual Book of ASTM Standards*, Section 4, Volume 04.02, Concrete and Mineral Aggregates, copyright American Society for Testing and Materials, 1916 Race Street, Philadelphia, PA 19103.

Included in this section:

ASTM C 29-87	Unit Weight and Voids in Aggregate
ASTM C 31-88	Making and Curing Concrete Test Specimens in the Field
ASTM C 94-86b	Standard Specification for Ready-Mixed Concrete
ASTM C 138-81	Unit Weight, Yield, and Air Content (Gravimetric) of Concrete
ASTM C 143-78	Slump of Portland Cement Concrete
ASTM C 172-82	Sampling Freshly Mixed Concrete
ASTM C 173-78	Air Content of Freshly Mixed Concrete by the Volumetric Method
ASTM C 231-82	Air Content of Freshly Mixed Concrete by the Pressure Method
ASTM C 1064-86	Temperature of Freshly Mixed Portland Cement Concrete
ASTM E 329-77 (Reapproved 1983)	Standard Recommended Practice for Inspection and Testing Agencies for Concrete, Steel, and Bituminous Materials as Used in Construction

Designation: C 29 – 87

Standard Test Method for Unit Weight and Voids in Aggregate[1]

This standard is issued under the fixed designation C 29; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This test method has been approved for use by agencies of the Department of Defense and for listing in the DoD Index of Specifications and Standards.

1. Scope

1.1 This test method covers the determination of unit weight in a compacted or loose condition and calculated voids in fine, coarse, or mixed aggregates based on the same determination. This test method is applicable to aggregates not exceeding 4 in. (100 mm) in nominal maximum size.

NOTE 1—Unit weight is the traditional terminology used to describe the property determined by this test method. Some believe the proper term is unit mass, or density, or bulk density, but consensus on this alternate terminology has not been obtained.

1.2 The values stated in inch-pound units are to be regarded as the standard except in regard to sieve sizes and the size of aggregate which are given in SI units in accordance with Specification E 11.

1.3 For other units of measure, the values stated in either inch-pound units or acceptable metric units are to be regarded separately as standard. The values stated in each system are not exact equivalents; therefore each system must be used independently of the other.

1.4 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 127 Test Method for Specific Gravity and Absorption of Coarse Aggregate[2]

C 128 Test Method for Specific Gravity and Absorption of Fine Aggregate[2]

C 670 Practice for Preparing Precision and Bias Statements for Construction Materials[2]

C 702 Methods for Reducing Field Samples of Aggregate to Testing Size[2]

D 75 Practice for Sampling Aggregates[3]

E 11 Specification for Wire-Cloth Sieves for Testing Purposes[4]

2.2 *AASHTO Standard:*

T19 Method for Unit Weight and Voids in Aggregates[5]

3. Significance and Use

3.1 This test method is often used to determine unit weight values that are necessary for use for many methods of selecting proportions for concrete mixtures.

3.2 The unit weight may also be used for determining mass/volume relationships for conversions in purchase agreements. However, the relationship between degree of compaction of aggregates in a hauling unit or stockpile and that achieved in this test method is unknown. Further, aggregates in hauling units and stockpiles usually contain absorbed and surface moisture (the latter affecting bulking), while this test method determines the unit weight on a dry basis.

3.3 A procedure is included for computing the percentage of voids between the aggregate particles based on the unit weight determined by this test method.

4. Apparatus

4.1 *Balance*—A balance or scale accurate within 0.1 % of the test load at any point within the range of use, graduated to at least 0.1 lb or 0.05 kg. The range of use shall be considered to extend from the weight of the measure empty to the weight of the measure plus its contents at 120 lb/ft^3 or 1920 kg/m^3.

4.2 *Tamping Rod*—A round, straight steel rod, ⅝ in. or 16 mm in diameter and approximately 24 in. or 600 mm in length, having one end rounded to a hemispherical tip of the same diameter as the rod.

4.3 *Measure*—A cylindrical metal measure, preferably provided with handles. It shall be watertight, with the top and bottom true and even; and sufficiently rigid to retain its form under rough usage. The measure shall have a height approximately equal to the diameter, but in no case shall the height be less than 80 % nor more than 150 % of the diameter. The capacity of the measure shall conform to the limits in Table 1 for the aggregate size to be tested. The thickness of metal in the measure shall be as described in Table 2. The top rim shall be smooth and plane within 0.01 in. or 0.25 mm and shall be parallel to the bottom within 0.5° (Note 2).

NOTE 2—The top rim is satisfactorily plane if a 0.01-in. or 0.25-mm feeler gage cannot be inserted between the rim and a piece of ¼-in. or 6-mm or thicker plate glass laid over the measure. The top and bottom are satisfactorily parallel if the slope between pieces of plate glass in contact with the top and bottom does not exceed 0.87 % in any direction.

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[1] This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.05 on Methods of Testing and Specifications for Physical Characteristics of Concrete Aggregates.

Current edition approved July 13, 1987. Published August 1987. Originally published as C 29 – 20 T. Last previous edition C 29 – 78. This edition of C 29 was extensively revised. It is recommended that users of this standard review this edition thoroughly.

[2] *Annual Book of ASTM Standards*, Vol 04.02.

[3] *Annual Book of ASTM Standards*, Vol 04.03.

[4] *Annual Book of ASTM Standards*, Vols 04.02 and 14.02.

[5] Available from American Association of State Highway and Transportation Officials, 444 N. Capitol St. NW, Suite 225, Washington, DC 20001.

TABLE 1 Capacity of Measures

Nominal Maximal Size of Aggregate		Capacity of Measure[A]	
in.	mm	ft³	L (m³)
½	(12.5)	¹⁄₁₀	2.8 (0.0028)
1	(25.0)	⅓	9.3 (0.0093)
1½	(37.5)	½	14 (0.014)
4	(100)	1	28 (0.028)

[A] The indicated size of measure may be used to test aggregates of a nominal maximum size equal to or smaller than that listed. The actual volume of the measure shall be at least 95 % of the nominal volume listed.

TABLE 2 Requirements for Measures

Capacity of Measure	Thickness of Metal, min		
	Bottom	Upper 1½ in. or 38 mm of wall[A]	Remainder of wall
Less than 0.4 ft³	0.20 in.	0.10 in.	0.10 in.
0.4 ft³ or more	0.20 in.	0.20 in.	0.12 in.
Less than 11 L	5.0 mm	2.5 mm	2.5 mm
11 L or more	5.0 mm	5.0 mm	3.0 mm

[A] The added thickness in the upper portion of the wall may be obtained by placing a reinforcing band around the top of the measure.

4.4 *Shovel or Scoop*—A shovel or scoop of convenient size for filling the measure with aggregate.

4.5 *Calibration Equipment*—A piece of plate glass, preferably at least ¼ in. or 6 mm thick and at least 1 in. or 25 mm larger than the diameter of the measure to be calibrated. A supply of water-pump or chassis grease that can be placed on the rim of the container to prevent leakage.

5. Sampling

5.1 Sampling should generally be accomplished in accordance with Practice D 75 and sample reduction in accordance with Methods C 702.

6. Test Sample

6.1 The size of the sample shall be approximately 125 to 200 % of the weight required to fill the measure, and shall be handled in a manner to avoid segregation. Dry the aggregate sample to essentially constant weight, preferably in an oven at 230 ± 9°F or 110 ± 5°C.

7. Calibration of Measure

7.1 Fill the measure with water at room temperature and cover with a piece of plate glass in such a way as to eliminate bubbles and excess water.

7.2 Determine the weight of the water in the measure using the balance described in 4.1.

7.3 Measure the temperature of the water and determine its density from Table 3, interpolating if necessary.

7.4 Calculate the volume, V, of the measure by dividing the weight of the water required to fill the measure by its density. Alternatively, calculate the factor for the measure ($1/V$) by dividing the density of the water by the weight required to fill the measure.

NOTE 3—For the calculation of unit weight, the volume of the measure in acceptable metric units should be expressed in cubic metres, or the factor as $1/m^3$. However, for convenience the size of the measure may be expressed in litres.

TABLE 3 Density of Water

Temperature		lb/ft³	kg/m³
°F	°C		
60	15.6	62.366	999.01
65	18.3	62.336	998.54
70	21.1	62.301	997.97
(73.4)	(23.0)	(62.274)	(997.54)
75	23.9	62.261	997.32
80	26.7	62.216	996.59
85	29.4	62.166	995.83

7.5 Measures shall be recalibrated at least once a year or whenever there is reason to question the accuracy of the calibration.

8. Selection of Procedure

8.1 The shoveling procedure for loose unit weight shall be used only when specifically stipulated. Otherwise, the compact unit weight shall be determined by the rodding procedure for aggregates having a nominal maximum size of 37.5 mm or 1½ in. or less, or by the jigging procedure for aggregates having a nominal maximum size greater than 37.5 mm or 1½ in. and not exceeding 100 mm or 4 in.

9. Rodding Procedure

9.1 Fill the measure one-third full and level the surface with the fingers. Rod the layer of aggregate with 25 strokes of the tamping rod evenly distributed over the surface. Fill the measure two-thirds full and again level and rod as above. Finally, fill the measure to overflowing and rod again in the manner previously mentioned. Level the surface of the aggregate with the fingers or a straightedge in such a way that any slight projections of the larger pieces of the coarse aggregate approximately balance the larger voids in the surface below the top of the measure.

9.2 In rodding the first layer, do not allow the rod to strike the bottom of the measure forcibly. In rodding the second and third layers, use only enough force to cause the tamping rod to penetrate the previous layer of aggregate.

9.3 Determine the weight of the measure plus its contents, and the weight of the measure alone, and record the values to the nearest 0.1 lb or 0.05 kg.

10. Jigging Procedure

10.1 Fill the measure in three approximately equal layers as described in 9.1, compacting each layer by placing the measure on a firm base, such as a cement-concrete floor, raising the opposite sides alternately about 2 in. or 50 mm, and allowing the measure to drop in such a manner as to hit with a sharp, slapping blow. The aggregate particles, by this procedure, will arrange themselves in a densely compacted condition. Compact each layer by dropping the measure 50 times in the manner described, 25 times on each side. Level the surface of the aggregate with the fingers or a straightedge in such a way that any slight projections of the larger pieces of the coarse aggregate approximately balance the larger voids in the surface below the top of the measure.

10.2 Determine the weight of the measure plus its contents, and the weight of the measure alone, and record the values to the nearest 0.1 lb or 0.05 kg.

11. Shoveling Procedure

11.1 Fill the measure to overflowing by means of a shovel or scoop, discharging the aggregate from a height not to exceed 2 in. or 50 mm above the top of the measure. Exercise care to prevent, so far as possible, segregation of the particle sizes of which the sample is composed. Level the surface of the aggregate with the fingers or a straightedge in such a way that any slight projections of the larger pieces of the coarse aggregate approximately balance the larger voids in the surface below the top of the measure.

11.2 Determine the weight of the measure plus its contents, and the weight of the measure alone, and record the values to the nearest 0.1 lb or 0.05 kg.

12. Calculation

12.1 *Unit Weight*—Calculate the unit weight for the rodding, jigging, or shoveling procedure as follows:

$$M = (G - T)/V \quad (1)$$

or

$$M = (G - T) \times F \quad (2)$$

where:
M = unit weight of the aggregate, lb/ft^3 or kg/m^3,
G = weight of the aggregate plus the measure, lb or kg,
T = weight of the measure, lb or kg,
V = volume of the measure, ft^3 or m^3, and
F = factor for measure, ft^{-3} or m^{-3}.

12.1.1 The unit weight determined by this test method is for aggregate in an oven-dry condition. If the unit weight in terms of saturated-surface-dry (SSD) condition is desired, use the exact procedure in this test method, and then calculate the SSD unit weight using the following formula:

$$M_{SSD} = M[1 + (A/100)] \quad (3)$$

where:
M_{SSD} = unit weight in SSD condition, lb/ft^3 or kg/m^3, and
A = % absorption, determined in accordance with Test Method C 127 or Test Method C 128.

12.2 *Void Content*—Calculate the void content in the aggregate using the unit weight determined by either the rodding, jigging, or shoveling procedure, as follows:

$$\% \text{ Voids} = [(S \times W) - M/S \times W]100 \quad (4)$$

where:
M = unit weight of the aggregate, lb/ft^3 or kg/m^3,
S = bulk specific gravity (dry basis) as determined in accordance with Test Method C 127 or Test Method C 128, and
W = density of the water, 62.3 lb/ft^3 or 998 kg/m^3.

13. Report

13.1 Report the results for the unit weight to the nearest 1 lb/ft^3 or 10 kg/m^3 as follows:

13.1.1 Unit weight by rodding, or

13.1.2 Unit weight by jigging, or

13.1.3 Loose unit weight.

13.2 Report the results for the void content to the nearest 1 % as follows:

13.2.1 Voids in aggregate compacted by rodding, %, or

13.2.2 Voids in aggregate compacted by jigging, %, or

13.2.3 Voids in loose aggregate, %.

14. Precision and Bias

14.1 The following estimates of precision for this test method are based on results from the AASHTO Materials Reference Laboratory (AMRL) Reference Sample Program, with testing conducted using this test method and AASHTO Method T19. There are no significant differences between the two test methods. The data are based on the analyses of more than 100 paired test results from 40 to 100 laboratories.

14.2 *Coarse Aggregate* (*unit weight*):

14.2.1 *Single-Operator Precision*—The single-operator standard deviation has been found to be 0.88 lb/ft^3 or 14 kg/m^3 (1S). Therefore, results of two properly conducted tests by the same operator on similar material should not differ by more than 2.5 lb/ft^3 or 40 kg/m^3 (D2S).

14.2.2 *Multilaboratory Precision*—The multilaboratory standard deviation has been found to be 1.87 lb/ft^3 or 30 kg/m^3 (1S). Therefore, results of two properly conducted tests from two different laboratories on similar material should not differ by more than 5.3 lb/ft^3 or 85 kg/m^3 (D2S).

14.2.3 These numbers represent, respectively, the (1S) and (D2S) limits as described in Practice C 670. The precision estimates were obtained from the analysis of AMRL reference sample data for unit weight (unit mass) by rodding of normal weight aggregates having a nominal maximum aggregate size of 25 mm or 1 in., and using a ½-ft^3 (14-L) measure.

14.3 *Fine Aggregate* (*unit weight*):

14.3.1 *Single-Operator Precision*—The single-operator standard deviation has been found to be 0.88 lb/ft^3 or 14 kg/m^3 (1S). Therefore, results of two properly conducted tests by the same operator on similar material should not differ by more than 2.5 lb/ft^3 or 40 kg/m^3 (D2S).

14.3.2 *Multilaboratory Precision*—The multilaboratory standard deviation has been found to be 2.76 lb/ft^3 or 44 kg/m^3 (1S). Therefore, results of two properly conducted tests from two different laboratories on similar material should not differ by more than 7.8 lb/ft^3 or 125 kg/m^3 (D2S).

14.3.3 These numbers represent, respectively, the (1S) and (D2S) limits as described in Practice C 670. The precision estimates were obtained from the analysis of AMRL reference sample data for loose unit weight (unit mass) from laboratories using a 1/10-ft^3 or 2.8-L measure.

14.4 No precision data on void content are available.

14.5 *Bias*—The procedure in this test method for measuring unit weight and void content has no bias because the values for unit weight and void content can be defined only in terms of a test method.

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Standard Practice for Making and Curing Concrete Test Specimens in the Field[1]

This standard is issued under the fixed designation C 31; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This practice has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

1. Scope

1.1 This practice covers procedures for making and curing cylindrical and prismatic specimens using job concrete that can be consolidated by rodding or vibration as described herein.

1.2 The concrete used to make the molded specimens shall have the same levels of slump, air content, and percentage of coarse aggregate as the concrete being placed in the work.

1.3 The values stated in inch-pound units are to be regarded as the standard.

1.4 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 143 Test Method for Slump of Portland Cement Concrete[2]
C 172 Method of Sampling Freshly Mixed Concrete[2]
C 173 Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method[2]
C 192 Method of Making and Curing Concrete Test Specimens in the Laboratory[2]
C 231 Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method[2]
C 470 Specification for Molds for Forming Concrete Test Cylinders Vertically[2]
C 511 Specification for Moist Cabinets, Moist Rooms, and Water Storage Tanks Used in the Testing of Hydraulic Cements and Concretes[2]
C 617 Practice for Capping Cylindrical Concrete Specimens[2]
C 1064 Test Methods for Temperature of Freshly Mixed Portland-Cement Concrete[2]

3. Significance and Use

3.1 This practice provides standardized requirements for making, curing, protecting, and transporting concrete test specimens under field conditions.

3.2 If specimen preparation is controlled as stipulated herein, the specimens may be used to develop information for the following purposes:

3.2.1 Checking the adequacy of laboratory mixture proportions for strength,

3.2.2 Serve as the basis for comparison with laboratory, field or in-place tests as the basis for safety and in-structure performance evaluation, and as the basis for form and shoring removal time requirements,

3.2.3 Determination of compliance with strength specifications, and

3.2.4 Determination of time when a structure may be put in service.

4. Apparatus

4.1 *Molds, General*—Molds for specimens or fastenings thereto in contact with the concrete shall be made of steel, cast iron, or other nonabsorbent material, nonreactive with concrete containing portland or other hydraulic cements. Molds shall hold their dimensions and shape under conditions of severe use. Molds shall be watertight during use as judged by their ability to hold water poured into them. Provisions for tests of watertightness are given in Section 6 of Specification C 470. A suitable sealant, such as heavy grease, modeling clay, or microcrystalline wax shall be used where necessary to prevent leakage through the joints. Positive means shall be provided to hold base plates firmly to the molds. Molds shall be lightly coated with mineral oil or a suitable nonreactive form release material before use.

4.2 *Cylinder Molds:*

4.2.1 *Molds for Casting Specimens Vertically*—Molds for casting concrete test specimens shall conform to the requirements of Specification C 470.

4.3 *Beam Molds*—Beam molds shall be rectangular in shape and of the dimensions required to produce the specimens stipulated in 5.2. The inside surfaces of the molds shall be smooth. The sides, bottom, and ends shall be at right angles to each other and shall be straight and true and free of warpage. Maximum variation from the nominal cross section shall not exceed 1/8 in. (3.2 mm) for molds with depth or breadth of 6 in. (152 mm) or more. Molds shall produce specimens not more than 1/16 in. (1.6 mm) shorter than the required length in accordance with 5.2, but may exceed it by more than that amount.

4.4 *Tamping Rod*—The rod shall be a round, straight steel rod 5/8 in. (16 mm) in diameter and approximately 24

[1] This practice is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.01 on Methods of Testing Concrete for Strength.
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[2] *Annual Book of ASTM Standards*, Vol 04.02.

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in. (610 mm) long, with the tamping end rounded to a hemispherical tip of the same diameter. Both ends may be rounded, if preferred.

4.5 *Vibrators*—Internal vibrators may have rigid or flexible shafts, preferably powered by electric motors. The frequency or vibration shall be 7000 vibrations per minute or greater while in use. The outside diameter or side dimension of the vibrating element shall be at least 0.75 in. (19 mm) and not greater than 1.50 in. (38 mm). The combined length of the shaft and vibrating element shall exceed the maximum depth of the section being vibrated by at least 3 in. (76 mm). When external vibrators are used, they should be the table or plank type. The frequency of external vibrators shall be at least 3600 vibrations per minute. For both table and plank vibrators, provision shall be made for clamping the mold securely to the apparatus. A vibrating-reed tachometer should be used to check the frequency of vibration.

4.6 *Mallet*—A mallet with a rubber or rawhide head weighing 1.25 ± 0.50 lb (0.57 ± 0.23 kg) shall be used.

4.7 *Small Tools*—Tools and items which may be required are shovels, pails, trowels, wood float, metal float, blunted trowels, straightedge, feeler gage, scoops, and rules.

4.8 *Slump Apparatus*—The apparatus for measurement of slump shall conform to the requirements of Test Method C 143.

4.9 *Sampling and Mixing Receptacle*—The receptacle shall be a suitable heavy gage metal pan, wheelbarrow, or flat, clean nonabsorbent mixing board of sufficient capacity to allow easy remixing of the entire sample with a shovel or trowel.

4.10 *Air Content Apparatus*—The apparatus for measuring air content shall conform to the requirements of Test Methods C 173 or C 231.

5. Test Specimens

5.1 *Compressive Strength Specimens*—Compressive strength specimens shall be cylinders of concrete cast and hardened in an upright position, with a length equal to twice the diameter. The standard specimen shall be the 6 by 12-in. (152 by 305-mm) cylinder when the maximum size of the coarse aggregate does not exceed 2 in. (50 mm). When the maximum size of the coarse aggregate does exceed 2 in. (50 mm), either the concrete sample shall be treated by wet sieving as described in Method C 172 or the diameter of the cylinder shall be at least three times the nominal maximum size of coarse aggregate in the concrete. Unless required by the project specifications, cylinders smaller than 6 of 12 in. shall not be made in the field.

NOTE 1—The maximum size is the smallest sieve opening through which the entire amount of aggregate is required to pass.

5.2 *Flexural Strength Specimens*—Flexural strength specimens shall be rectangular beams of concrete cast and hardened with long axes horizontal. The length shall be at least 2 in. (50 mm) greater than three times the depth as tested. The ratio of width to depth as molded shall not exceed 1.5. The standard beam shall be 6 by 6 in. (152 by 152 mm) in cross section, and shall be used for concrete with maximum size coarse aggregate up to 2 in. (50 mm). When the nominal maximum size of the coarse aggregate exceeds 2 in. (50 mm), the smaller cross sectional dimension of the beam shall be at least three times the nominal maximum size of the coarse aggregate. Unless required by project specifications, beams made in the field shall not have a width or depth of less than 6 in.

6. Sampling Concrete

6.1 The samples used to fabricate test specimens under this standard shall be obtained in accordance with Method C 172 unless an alternative procedure has been approved.

6.2 Record the identity of the sample with respect to the location of the concrete represented and the time of casting.

7. Slump, Air Content, and Temperature

7.1 *Slump*—Measure the slump of each batch of concrete, from which specimens are made, immediately after remixing in the receptacle, as required in Test Method C 143.

7.2 *Air Content*—Determine the air content in accordance with either Test Method C 173 or Test Method C 231. The concrete used in performing the air content test shall not be used in fabricating test specimens.

7.3 *Temperature*—Determine the temperature in accordance with Test Method C 1064.

8. Molding Specimens

8.1 *Place of Molding*—Mold specimens promptly on a level, rigid, surface, free of vibration and other disturbances, at a place as near as practicable to the location where they are to be stored.

8.2 *Placing the Concrete*—Place the concrete in the molds using a scoop, blunted trowel, or shovel. Select each scoopful, trowelful, or shovelful of concrete from the mixing pan to ensure that it is representative of the batch. Remix the concrete in the mixing pan with a shovel or trowel to prevent segregation during the molding of specimens. Move the scoop, trowel, or shovel around the perimeter of the mold opening when adding concrete to ensure an even distribution of the concrete and minimize segregation. Further distribute the concrete by use of a tamping rod prior to the start of consolidation. In placing the final layer the operator shall attempt to add an amount of concrete that will exactly fill the mold after compaction. Do not add nonrepresentative concrete to an underfilled mold.

8.2.1 *Number of Layers*—Make specimens in layers as indicated in Table 1.

8.3 *Consolidation:*

8.3.1 *Methods of Consolidation*—Preparation of satisfactory specimens requires different methods of consolidation. The methods of consolidation are rodding, and internal or external vibration. Base the selection of the method of consolidation on the slump, unless the method is stated in the specifications under which the work is being performed. Rod concretes with a slump greater than 3 in. (75 mm). Rod or vibrate concretes with slump of 1 to 3 in. (25 to 75 mm). Vibrate concretes with slump of less than 1 in. (25 mm). Concretes of such low water content that they cannot be properly consolidated by the methods described herein, or requiring other sizes and shapes of specimens to represent the product or structure, are not covered by this method. Specimens for such concretes shall be made in accordance with the requirements of Method C 192 with regard to specimen size and shape and method of consolidation.

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TABLE 1 Number of Layers Required for Specimens

Specimen Type and Size, as Depth, in. (mm)	Mode of Compaction	Number of Layers	Approximate Depth of Layer, in. (mm)
Cylinders:			
12 (305)	rodding	3 equal	4 (100)
Over 12 (305)	rodding	as required	4 (100)
12 (305) to 18 (460)	vibration	2 equal	half depth of specimens
Over 18 (460)	vibration	3 or more	8 (200) as near as practicable
Beams:			
6 (152) to 8 (200)	rodding	2 equal	half depth of specimen
Over 8 (200)	rodding	3 or more	4 (100)
6 (152) to 8 (200)	vibration	1	depth of specimen
Over 8 (200)	vibration	2 or more	8 (200) as near as practicable

TABLE 2 Number of Roddings to be Used in Molding Cylinder Specimens

Diameter of Cylinder, in. (mm)	Number of Strokes/Layer
6 (152)	25
8 (200)	50
10 (250)	75

8.3.2 *Rodding*—Place the concrete in the mold, in the required number of layers of approximately equal volume. For cylinders, rod each layer with the rounded end of the rod using the number of strokes specified in Table 2. The number of roddings per layer required for beams is one for each 2-in.^{2} (13-cm^2) top surface area of the specimen. Rod the bottom layer throughout its depth. Distribute the strokes uniformly over the cross section of the mold and for each upper layer allow the rod to penetrate about ½ in. (12 mm) into the underlying layer when the depth of the layer is less than 4 in. (100 mm), and about 1 in. (25 mm) when the depth is 4 in. or more. After each layer is rodded, tap the outsides of the mold lightly 10 to 15 times with the mallet, to close any holes left by rodding and to release any large air bubbles that may have been trapped. Tap light-gage single-use molds, susceptible to damage if tapped with the mallet, using an open hand. After tapping, spade the concrete along the sides and ends of beam molds with a trowel or other suitable tool.

8.3.3 *Vibration*—Maintain a uniform time period for duration of vibration for the particular kind of concrete, vibrator, and specimen mold involved. The duration of vibration required will depend upon the workability of the concrete and the effectiveness of the vibrator. Usually sufficient vibration has been applied as soon as the surface of the concrete has become relatively smooth. Continue vibration only long enough to achieve proper consolidation of the concrete. Overvibration may cause segregation. Fill the molds and vibrate in the required number of approximately equal layers. Place all the concrete for each layer in the mold before starting vibration of that layer. When placing the final layer, avoid overfilling by more than ¼ in. (6 mm). Finish the surface either during or after vibration where external vibration is used. Finish the surface after vibration when internal vibration is used. When the finish is applied after vibration, add only enough concrete with a trowel to overfill the mold about ⅛ in. (3 mm). Work it into the surface and then strike it off.

8.3.3.1 *Internal Vibration*—The diameter of the vibrating element, or thickness of a square vibrating element, shall be in accordance with the requirements of 4.5. For beams, the vibrating element shall not exceed ⅓ of the width of the mold. For cylinders, the ratio of the diameter of the cylinder to the diameter of the vibrating element shall be 4.0 or higher. In compacting the specimen the vibrator shall not be allowed to rest on the bottom or sides of the mold. Carefully withdraw the vibrator in such a manner that no air pockets are left in the specimen.

8.3.3.2 *Cylinders*—Use three insertions of the vibrator at different points for each layer. Allow the vibrator to penetrate through the layer being vibrated, and into the layer below, approximately 1 in. (25 mm). After each layer is vibrated, tap the outsides of the mold lightly 10 to 15 times with the mallet, to close any holes left by rodding and to release any large air bubbles that may have been trapped. Tap light-gage single-use molds, susceptible to damage if tapped with the mallet, using an open hand.

8.3.3.3 *Beam*—Insert the vibrator at intervals not exceeding 6 in. (150 mm) along the center line of the long dimension of the specimen. For specimens wider than 6 in., use alternating insertions along two lines. Allow the shaft of the vibrator to penetrate into the bottom layer approximately 1 in. (25 mm). After each layer is vibrated, tap the outsides of the mold lightly 10 to 15 times with the mallet to close any holes left by vibrating and to release any large air bubbles that may have been trapped.

8.3.4 *External Vibration*—When external vibration is used, take care to ensure that the mold is rigidly attached to or securely held against the vibrating element or vibrating surface.

8.4 *Finishing*—After consolidation, unless the finishing has been performed during the vibration (8.3.3), strike off the surface of the concrete and float or trowel it as required. Perform all finishing with the minimum manipulation necessary to produce a flat even surface that is level with the rim or edge of the mold and that has no depressions or projections larger than ⅛ in. (3.2 mm).

8.4.1 *Cylinders*—After consolidation, finish the top surfaces by striking them off with the tamping rod where the consistency of the concrete permits or with a wood float or trowel. If desired, cap the top surface of freshly made cylinders with a thin layer of stiff portland cement paste which is permitted to harden and cure with the specimen. See section on Capping Materials of Practice C 617.

8.4.2 *Beams*—After consolidation of the concrete, strike off the top surface to the required tolerance to produce a flat even surface. A wood float may be used.

8.5 *Initial Storage*—Immediately after being struck off, the specimens shall be moved to the storage place where they will remain undisturbed for the initial curing period. If specimens made in single use mold are moved, lift and support the specimens from the bottom of the molds with a large trowel or similar device.

9. Curing

9.1 *Covering After Finishing*—Immediately after finishing, precautions shall be taken to prevent evaporation and loss of water from the specimens. Protect the outside surfaces of cardboard molds from contact with wet burlap or other sources of water. Cardboard molds may expand and damage specimens at an early age if the outside of the mold absorbs water. Cover specimens with a nonabsorbent, nonreactive plate or sheet of impervious plastic. Wet burlap may be used over the plate or plastic sheet to help retard evaporation, but the burlap must not be in contact with the surface of the concrete.

9.2 *Curing Specimens for Checking the Adequacy of Laboratory Mixture Proportions for Strength or as the Basis for Acceptance or Quality Control:*

9.2.1 *Initial Curing:*

9.2.1.1 *Initial Curing in Air*—During the initial 24 ± 8 h after molding, the temperature immediately adjacent to the specimens shall be maintained in the range of 60 to 80°F (16 to 27°C), and loss of moisture from the specimens shall be prevented (Note 2). Temperature differentials in and between specimens shall be controlled by shielding from the direct rays of the sun and from radiant heating devices. Specimens not to be transported shall be removed from the molds after the initial 24 ± 8 h and standard curing shall be started as required by 9.2.2. Specimens to be transported prior to 48 h after molding shall not be demolded, but shall continue initial curing at 60 to 80°F (16 to 27°C) until time for transporting. Specimens to be transported after 48-h age shall be demolded in 24 ± 8 h. Curing shall then be continued but in saturated limewater at 73.4 ± 3°F (23 ± 1.7°C) until the time of transporting.

NOTE 2—It may be necessary to create an environment during the initial curing in air period to provide satisfactory moisture conditions and to control the temperature in the range of 60 to 80°F (16 to 27°C). The specimens may be stored in tightly constructed wooden boxes, damp sand pits, temporary buildings at construction sites, under wet burlap in favorable weather or in heavyweight closed plastic bags, or use other suitable methods, provided the foregoing requirements limiting specimen temperature and moisture loss are met. The temperature may be controlled by ventilation, or thermostatically controlled cooling devices, or by heating devices such as stoves, light bulbs or thermostatically controlled heating elements. Temperature record of the specimens may be established by means of maximum-minimum thermometers.

9.2.1.2 *Initial Curing of Cylinders in Water*—Immediately after molding, immerse the specimens in saturated limewater at 60 to 80°F (16 to 27°C) for 24 ± 8 h. This curing is not acceptable for specimens in cardboard molds or molds which expand when immersed in water. Remove specimens from molds at 24 ± 8 h, protect from loss of moisture, and within 30 min start standard curing at 73.4 ± 3°F (23 ± 1.7°C) as required in 9.2.2.

9.2.2 *Standard Curing:*

9.2.2.1 *Cylinders*—Upon completion of initial curing and within 30 min after removing the molds, store specimens in a moist condition with free water maintained on their surfaces at all times at a temperature of 73.4 ± 3°F (23 ± 1.7°C). Temperatures between 68 and 86°F (20 and 30°C) are permitted for a period not to exceed 3 h immediately prior to test if free moisture is maintained on the surfaces of the specimen at all times, except when capping with sulfur mortar capping compound. When capping with this material, the ends of the cylinder will be dried as described in Method C 617. Specimens shall not be exposed to dripping or running water. The required moist storage can be obtained by immersion in saturated limewater and may be obtained by storage in a moist room or cabinet meeting the requirements of Specification C 511.

9.2.2.2 *Beams*—Beams are to be cured the samed as cylinders, see 9.2.2.1, except for a minimum of 20 h prior to testing, they shall be stored in saturated limewater at 73.4 ± 3°F (23 ± 1.7°C). Drying of the surfaces of the beam shall be prevented between removal from limewater and completion of testing.

NOTE 3—Relatively small amounts of surface drying of flexural specimens can induce tensile stresses in the extreme fibers that will markedly reduce the indicated flexural strength.

9.3 *Curing for Determining Form Removal Time or When a Structure May be Put into Service:*

9.3.1 *Cylinders*—Store cylinders in or on the structure as near to the point of deposit of the concrete represented as possible. Protect all surfaces of the cylinders from the elements in as near as possible the same way as the formed work. Provide the cylinders with the same temperature and moisture environment as the structural work. Test the specimens in the moisture condition resulting from the specified curing treatment. To meet these conditions, specimens made for the purpose of determining when a structure may be put in service shall be removed from the molds at the time of removal of form work.

9.3.2 *Beams*—As nearly as practicable, cure beams in the same manner as the concrete in the structure. At the end of 48 ± 4 h after molding, take the molded specimens to the storage location and remove from the molds. Store specimens representing pavements of slabs on grade by placing them on the ground as molded, with their top surfaces up. Bank the sides and ends of the specimens with earth or sand that shall be kept damp, leaving the top surfaces exposed to the specified curing treatment. Store specimens representing structure concrete as near the point in the structure they represent as possible, and afford them the same temperature protection and moisture environment as the structure. At the end of the curing period leave the specimens in place exposed to the weather in the same manner as the structure. Remove all beam specimens from field storage and store in limewater at 73.4 ± 5°F (23 ± 2.8°C) for 24 ± 4 h immediately before time of testing to ensure uniform moisture condition from specimen to specimen. Observe the precautions given in 9.2.2.2 to guard against drying between time of removal from curing to testing.

10. Transportation of Specimens to Laboratory

10.1 Specimens shall not be transported from the field to the laboratory before completion of the initial curing. Specimens to be transported prior to an age of 48 h shall not be demolded prior to completion of transportation. Prior to transporting, specimens shall be cured and protected as required in Section 9. During transportation, the specimens must be protected with suitable cushioning material to prevent damage from jarring and from damage by freezing temperatures, or moisture loss. Moisture loss may be prevented by wrapping the specimens in plastic or surrounding them with wet sand or wet saw dust. When specimens are received by the laboratory, they shall be removed from molds if not done before shipment and placed in the required standard curing at 73.4 ± 3°F (23 ± 1.7°C).

Designation: C 94 – 86b

Standard Specification for Ready-Mixed Concrete[1]

This standard is issued under the fixed designation C 94; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This specification has been approved for use by agencies of the Department of Defense and for listing in the DoD Index of Specifications and Standards.

1. Scope

1.1 This specification covers ready-mixed concrete manufactured and delivered to a purchaser in a freshly mixed and unhardened state as hereinafter specified. Requirements for quality of concrete shall be either as hereinafter specified or as specified by the purchaser. In any case where the requirements of the purchaser differ from these in this specification, the purchaser's specification shall govern. This specification does not cover the placement, consolidation, curing, or protection of the concrete after delivery to the purchaser.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 As used throughout this specification the manufacturer shall be the contractor, subcontractor, supplier, or producer who furnishes the ready-mixed concrete. The purchaser shall be the owner or representative thereof.

2. Referenced Documents

2.1 *ASTM Standards:*

C 31 Practice for Making and Curing Concrete Test Specimens in the Field[2]

C 33 Specification for Concrete Aggregates[2]

C 39 Test Method for Compressive Strength of Cylindrical Concrete Specimens[2]

C 109 Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm Cube Specimens)[3]

C 138 Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete[2]

C 143 Test Method for Slump of Portland Cement Concrete[2]

C 150 Specification for Portland Cement[2]

C 172 Method of Sampling Freshly Mixed Concrete[2]

C 173 Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method[2]

C 191 Test Method for Time of Setting of Hydraulic Cement by Vicat Needle[3]

C 231 Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method[2]

C 260 Specification for Air Entraining Admixtures for Concrete[2]

C 330 Specification for Lightweight Aggregates for Structural Concrete[2]

C 494 Specification for Chemical Admixtures for Concrete[2]

C 567 Test Method for Unit Weight of Structural Lightweight Concrete[2]

C 595 Specification for Blended Hydraulic Cements[2]

C 618 Specification for Fly Ash and Raw or Calcined Natural Pozzolans for Use as a Mineral Admixture in Portland Cement Concrete[2]

C 989 Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete Mortars[2]

C 1017 Specification for Chemical Admixtures for Use in Producing Flowing Concrete[2]

D 512 Test Methods for Chloride Ion in Water[4]

D 516 Test Methods for Sulfate Ion in Water[4]

E 329 Recommended Practice for Inspection and Testing Agencies for Concrete, Steel, and Bituminous Materials as Used in Construction[5]

2.2 *American Concrete Institute Standards:*[6]

CP-2 Concrete Field Testing Technician, Grade I

211.1 Recommended Practice for Selecting Proportions for Normal and Heavyweight Concrete

211.2 Recommended Practice for Selecting Proportions for Structural Lightweight Concrete

214 Practice for Evaluation of Strength Test Results of Concrete

305R Hot Weather Concreting

306R Cold Weather Concreting

2.3 *National Bureau of Standards Document:*[7]

Handbook 44 Specifications, Tolerances, and other Technical Requirements for Commercial Weighing and Measuring Devices

2.4 Other Documents:

Bureau of Reclamation Concrete Manual[7]

AASHTO T 26 Method of Test for Quality of Water to be Used in Concrete[8]

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[1] This specification is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.09 on Methods of Testing and Specifications for Ready-Mixed Concrete.

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[2] *Annual Book of ASTM Standards*, Vol 04.02.

[3] *Annual Book of ASTM Standards*, Vol 04.01.

[4] *Annual Book of ASTM Standards*, Vol 11.01.

[5] *Annual Book of ASTM Standards*, Vol 14.02.

[6] Available from American Concrete Institute, P.O. Box 19150, Detroit, MI 48219.

[7] Available at Superintendent of Documents, U. S. Government Printing Office, Washington, DC 20402.

[8] Available from the American Association of State Highway and Transportation Officials, 444 N. Capitol St., NW, Suite 225, Washington, DC 20001.

3. Basis of Purchase

3.1 The basis of purchase shall be the cubic yard or cubic metre of freshly mixed and unhardened concrete as discharged from the mixer.

3.2 The volume of freshly mixed and unhardened concrete in a given batch shall be determined from the total weight of the batch divided by the actual weight per cubic foot of the concrete. The total weight of the batch shall be calculated either as the sum of the weights of all materials, including water, entering the batch or as the net weight of the concrete in the batch as delivered. The weight per cubic foot shall be determined in accordance with Test Method C 138 from the average of at least three measurements, each on a different sample using a ½-ft^3 (14 160-cm^3) container. Each sample shall be taken from the midpoint of each of three different truck loads by the procedure outlined in Method C 172.

NOTE 1—It should be understood that the volume of hardened concrete may be, or appear to be, less than expected due to waste and spillage, over-excavation, spreading forms, some loss of entrained air, or settlement of wet mixtures, none of which are the responsibility of the producer.

4. Materials

4.1 In the absence of designated applicable specifications covering requirements for quality of materials, the following specifications shall govern:

4.1.1 *Cement*—Cement shall conform to Specification C 150 or Specification C 595. The purchaser should specify the type or types required, but if no type is specified, the requirements of Type I as prescribed in Specification C 150 shall apply.

NOTE 2—These different cements will produce concretes of different properties and should not be used interchangeably.

4.1.2 *Aggregates*—Aggregates shall conform to Specification C 33 or Specification C 330 if lightweight concrete is specified by the purchaser.

4.1.3 *Water:*

4.1.3.1 The mixing water shall be clear and apparently clean. If it contains quantities of substances which discolor it or make it smell or taste unusual or objectionable or cause suspicion, it shall not be used unless service records of concrete made with it or other information indicates that it is not injurious to the quality of the concrete. Water of questionable quality shall be subject to the acceptance criteria of Table 1.

4.1.3.2 Wash water from mixer washout operations may be used for mixing concrete provided tests of wash water comply with the physical tests of Table 1. Wash water shall be tested at a weekly interval for approximately 4 weeks, and thereafter at a monthly interval provided no single test exceeds the applicable limit (Note 3). Optional chemical tests in Table 2 may be specified by the purchaser when appropriate for the construction. The testing frequency for chemical limits should be as given above or as specified by the purchaser.

NOTE 3—When recycled wash water is used, attention should be given to effects on the dosage rate and batching sequence of air-entraining and other chemical admixtures, and a uniform amount should be used in consecutive batches.

4.1.4 *Admixtures*—Admixtures shall conform to Specifications C 260, C 494, C 618, C 989, and C 1017, if applicable.

NOTE 4—In any given instance, the required dosage of air-entraining, accelerating, and retarding admixtures will vary. Therefore, a range of dosages should be allowed which will permit obtaining the desired effect.

5. Ordering Information

5.1 In the absence of designated applicable general specifications, the purchaser shall specify the following:

5.1.1 Designated size, or sizes, of coarse aggregate,

5.1.2 Slump, or slumps, desired at the point of delivery (see Section 6 for acceptable tolerances),

5.1.3 When air-entrained concrete is specified, the air content of the samples taken at the point of discharge from the transportation unit (see Section 7 and Table 3 for the total air content and tolerances) (Note 4).

5.1.4 Which of Alternatives 1, 2, or 3 shall be used as a basis for determining the proportions of the concrete to produce the required quality, and

5.1.5 When structural lightweight concrete is specified, the unit weight as wet weight, air-dry weight, or oven-dry weight (Note 6).

NOTE 5—In selecting the specified air content, the purchaser should consider the exposure conditions to which the concrete will be subjected. Air contents less than shown in Table 3 may not give the required resistance to freezing and thawing, which is the primary purpose of air-entrained concrete. Air contents higher than the levels shown may reduce strength without contributing any further improvement of durability.

NOTE 6—The unit weight of fresh concrete, which is the only unit weight determinable at the time of delivery, is always higher than the air-dry or oven-dry weight. Definitions of, and methods for determining or calculating air-dry and oven-dry weights, are covered by Test Method C 567.

5.2 *Alternative No. 1:*

5.2.1 When the purchaser assumes responsibility for the proportioning of the concrete mixture, he shall also specify the following:

5.2.1.1 Cement content in bags or pounds per cubic yard of concrete, or equivalent units,

5.2.1.2 Maximum allowable water content in gallons per cubic yard of concrete, or equivalent units, including surface moisture on the aggregates, but excluding water of absorption (Note 7), and

5.2.1.3 If admixtures are required, the type, name, and dosage to be used. The cement content shall not be reduced when admixtures are used under Alternative No. 1 without the written approval of the purchaser.

NOTE 7—The purchaser, in selecting requirements for which he assumes responsibility should give consideration to requirements for workability, placeability, durability, surface texture, and density, in addition to those for structural design. The purchaser is referred to American Concrete Institute Standard 211.1 and American Concrete Institute Standard 211.2 for the selection of proportions that will result in concrete suitable for various types of structures and conditions of exposure. The water-cement ratio of most structural lightweight concretes cannot be determined with sufficient accuracy for use as a specification basis.

5.2.2 At the request of the purchaser, the manufacturer shall, prior to the actual delivery of the concrete, furnish a statement to the purchaser giving the sources, specific

TABLE 1 Acceptance Criteria for Questionable Water Supplies

	Limits	Test Method
Compressive strength, min % control at 7 days	90	C 109[A]
Time of set, deviation from control, h: min	from 1:00 early to 1:30 later	C 191[A]

[A] Comparisons shall be based on fixed proportions and the same volume of test water compared to control mix using city water or distilled water.

TABLE 2 Chemical Limitations for Wash Water

	Limits	Test Method[A]
Chemical requirements, maximum concentration in mixing water, ppm[B]		
Chloride as Cl, ppm:		D 512
Prestressed concrete or in bridge decks	500[C]	
Other reinforced concrete in moist environments or containing aluminum embedments or dissimilar metals or with stay-in-place galvanized metal forms	1000[C]	
Sulfate as SO_4, ppm	3000	D 516
Alkalies as (Na_2O + 0.658 K_2O), ppm	600	
Total solids, ppm	50 000	AASHTO T26

[A] Other test methods that have been demonstrated to yield comparable results may be used.
[B] Wash water reused as mixing water in concrete may exceed the listed concentrations of chloride and sulfate if it can be shown that the concentration calculated in the total mixing water, including mixing water on the aggregates and other sources does not exceed the stated limits.
[C] For conditions allowing use of $CaCl_2$ accelerator as an admixture, the chloride limitation may be waived by the purchaser.

gravities, and sieve analyses of the aggregates and the dry weights of cement and saturated-surface-dry weights of fine and coarse aggregate and quantities, type and name of admixture (if any) and of water per cubic yard or cubic metre of concrete that will be used in the manufacture of each class of concrete ordered by the purchaser.

5.3 *Alternative No. 2:*

5.3.1 When the purchaser requires the manufacturer to assume full responsibility for the selection of the proportions for the concrete mixture (Note 7), the purchaser shall also specify the following:

5.3.1.1 Requirements for compressive strength as determined on samples taken from the transportation unit at the point of discharge evaluated in accordance with Section 17. The purchaser shall specify the requirements in terms of the compressive strength of standard specimens cured under standard laboratory conditions for moist curing (see Section 19). Unless otherwise specified the age at test shall be 28 days.

5.3.2 At the request of the purchaser, the manufacturer shall, prior to the actual delivery of the concrete, furnish a statement to the purchaser, giving the dry weights of cement and saturated surface-dry-weights of fine and coarse aggregate and quantities, type, and name of admixtures (if any) and of water per cubic yard or cubic metre of concrete that will be used in the manufacture of each class of concrete ordered by the purchaser. He shall also furnish evidence satisfactory to the purchaser that the materials to be used and proportions selected will produce concrete of the quality specified.

5.4 *Alternative No. 3:*

5.4.1 When the purchaser requires the manufacturer to assume responsibility for the selection of the proportions for the concrete mixture with the minimum allowable cement content specified (Note 7), the purchaser shall also specify the following:

5.4.1.1 Required compressive strength as determined on samples taken from the transportation unit at the point of discharge evaluated in accordance with Section 17. The purchaser shall specify the requirements for strength in terms of tests of standard specimens cured under standard laboratory conditions for moist curing (see Section 19). Unless otherwise specified the age at test shall be 28 days.

5.4.1.2 Minimum cement content in bags or pounds per cubic yard or kilograms per cubic metre of concrete.

5.4.1.3 If admixtures are required, the type, name, and dosage to be used. The cement content shall not be reduced when admixtures are used.

NOTE 8—Alternative No. 3 can be distinctive and useful only if the designated minimum cement content is at about the same level that would ordinarily be required for the strength, aggregate size, and slump specified. At the same time, it must be an amount that will be sufficient to assure durability under expected service conditions, as well as satisfactory surface texture and density, in the event specified strength is attained with it. For additional information refer to ACI Standards 211.1 and 211.2 referred to in Note 7.

5.4.2 At the request of the purchaser, the manufacturer shall, prior to the actual delivery of the concrete, furnish a statement to the purchaser, giving the dry weights of cement and saturated surface-dry weights of fine and coarse aggregate and quantities, type, and name of admixture (if any) and of water per cubic yard or cubic metre of concrete that will be used in the manufacture of each class of concrete ordered by the purchaser. He shall also furnish evidence satisfactory to the purchaser that the materials to be used and proportions selected will produce concrete of the quality specified. Whatever strengths are attained the quantity of cement used shall not be less than the minimum specified.

5.5 The proportions arrived at by Alternatives 1, 2, or 3 for each class of concrete and approved for use in a project shall be assigned a designation to facilitate identification of each concrete mixture delivered to the project. This is the designation required in 16.1.7 and supplies information on concrete proportions when they are not given separately on each delivery ticket as outlined in 16.2. A certified copy of all proportions as established in Alternatives 1, 2, and 3 shall be on file at the batch plant.

TABLE 3 Recommended Total Air Content for Air-Entrained Concrete[A,C]

Exposure Condition[B]	Total Air Content, %						
	Nominal Maximum Sizes of Aggregate, in. (mm)						
	3/8 (9.5)	1/2 (12.5)	3/4 (19.0)	1 (25.0)	1½ (37.5)	2 (50.0)	3 (75.0)
Mild	4.5	4.0	3.5	3.0	2.5	2.0	1.5
Moderate	6.0	5.5	5.0	4.5	4.5	4.0	3.5
Severe	7.5	7.0	6.0	6.0	5.5	5.0	4.5

[A] For air-entrained concrete, when specified.
[B] For description of exposure conditions, refer to ACI 211.1, Table 5.3.3, with attention to accompanying footnotes.
[C] Unless exposure conditions dictate otherwise, air contents recommended above may be reduced by up to 1 % for concretes with specified compressive strength, f'_c, of 5000 psi (34.5 MPa) or above.

6. Tolerances in Slump

6.1 Unless other tolerances are included in the project specifications, the following shall apply.

6.1.1 When the project specifications for slump are written as a "maximum" or "not to exceed" requirement:

	Specified slump:	
	If 3 in. (76 mm) or less	If more than 3 in. (76 mm)
Plus tolerance:	0	0
Minus tolerance:	1½ in. (38 mm)	2½ in. (63 mm)

This option is to be used only if one addition of water is permitted on the job provided such addition does not increase the water-cement ratio above the maximum permitted by the specifications.

6.1.2 When the project specifications for slump are *not* written as a "maximum" or "not to exceed" requirement:

Tolerances for Nominal Slumps

For Specified Slump of:	Tolerance
2 in. (51 mm) and less	±½ in. (13 mm)
More than 2 through 4 in. (51 to 102 mm)	±1 in. (25 mm)
More than 4 in. (102 mm)	±1½ in. (38 mm)

6.2 Concrete shall be available within the permissible range of slump for a period of 30 min starting either on arrival at the job site or after the initial slump adjustment permitted in 11.7, whichever is later. The first and last ¼ yd³ or ¼ m³ discharged are exempt from this requirement. If the user is unprepared for discharge of the concretes from the vehicle, the producer shall not be responsible for the limitation of minimum slump after 30 min have elapsed starting either on arrival of the vehicle at the prescribed destination or at the requested delivery time, whichever is later.

7. Air-Entrained Concrete

7.1 When air-entrained concrete is desired the purchaser shall specify the total air content of the concrete. See Table 3 for recommended total air contents (Note 4).

7.2 The air content of air-entrained concrete when sampled from the transportation unit at the point of discharge shall be within a tolerance of ± 1.5 of the specified value.

8. Measuring Materials

8.1 Except as otherwise specifically permitted, cement shall be measured by weight. When fly ash or other pozzolans are specified in the mix design, they may be weighed cumulatively with cement. Cement and pozzolan shall be weighed on a scale and in a weigh hopper which is separate and distinct from those used for other materials. Cement shall be weighed before pozzolan. When the quantity of cement exceeds 30 % of the full capacity of the scale, the quantity of cement, and the cumulative quantity of cement plus pozzolan, shall be within ±1 % of the required weight. For smaller batches to a minimum of 1 yd³ (1 m³), the quantity of cement, and the quantity of cement plus pozzolan, used shall be not less than the required amount nor more than 4 % in excess. Under special circumstances, approved by the purchaser, cement may be measured in bags of standard weight (Note 9). No fraction of a bag of cement shall be used unless weighed.

NOTE 9—In the United States the standard weight of a bag of portland cement is 94 lb (42.6 kg) ±3 %.

8.2 Aggregate shall be measured by weight. Batch weights shall be based on dry materials and shall be the required weights of dry materials plus the total weight of moisture (both absorbed and surface) contained in the aggregate. The quantity of aggregate used in any batch of concrete as indicated by the scale shall be within ±2 % of the required weight when weighed in individual aggregate weigh batchers. In a cumulative aggregate weigh batcher, the cumulative weight after each successive weighing shall be within ±1 % of the required cumulative amount when the scale is used in excess of 30 % of its capacity. For cumulative weights for less than 30 % of scale capacity, the tolerance shall be ±0.3 % of scale capacity or ±3 % of the required cumulative weight, whichever is less.

8.3 Mixing water shall consist of water added to the batch, ice added to the batch, water occurring as surface moisture on the aggregates, and water introduced in the form of admixtures. The added water shall be measured by weight or volume to an accuracy of 1 % of the required total mixing water. Added ice shall be measured by weight. In the case of truck mixers, any wash water retained in the drum for use in the next batch of concrete shall be accurately measured; if this proves impractical or impossible the wash water shall be discharged prior to loading the next batch of concrete. Total water (including any wash water) shall be measured or weighed to an accuracy of ±3 % of the specified total amount.

8.4 Powdered admixtures shall be measured by weight, and paste or liquid admixtures by weight or volume. Accuracy of weighing admixtures shall be within ±3 % of the required weight. Volumetric measurement shall be within an accuracy of ±3 % of the total amount required or plus and minus the volume of dose required for one sack of cement, whichever is greater.

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NOTE 10—Admixture dispensers of the mechanical type capable of adjustment for variation of dosage, and of simple calibration, are recommended.

9. Batching Plant

9.1 Bins with adequate separate compartments shall be provided in the batching plant for fine and for each required size of coarse aggregate. Each bin compartment shall be designed and operated so as to discharge efficiently and freely, with minimum segregation, into the weighing hopper. Means of control shall be provided so that, as the quantity desired in the weighing hopper is approached, the material may be shut off with precision. Weighing hoppers shall be constructed so as to eliminate accumulations of tare materials and to discharge fully.

9.2 Indicating devices shall be in full view and near enough to be read accurately by the operator while charging the hopper. The operator shall have convenient access to all controls.

9.3 Scales in use shall be accurate when static load tested to ±0.4 % of the total capacity of the scale.

9.4 Scales for batching concrete ingredients may be either beam or springless dial scales and shall conform to the applicable sections of the current edition of the National Bureau of Standards Handbook 44, except as may be otherwise specified. Methods for weighing (electric, hydraulic, load cells, etc.) other than beam or springless dial scales which meet the above weighing tolerances are also acceptable.

9.5 Adequate standard test weights shall be available for checking accuracy. All exposed fulcrums, clevises, and similar working parts of scales shall be kept clean. Beam scales shall be equipped with a balance indicator sensitive enough to show movement when a weight equal to 0.1 % of the nominal capacity of the scale is placed in the batch hopper. Pointer travel shall be a minimum of 5 % of the net-rated capacity of the largest weigh beam for underweight and 4 % for overweight.

9.6 The device for the measurement of the added water shall be capable of delivering to the batch the quantity required within the accuracy required in 8.3. The device shall be so arranged that the measurements will not be affected by variable pressures in the water supply line. Measuring tanks shall be equipped with outside taps and valves to provide for checking their calibration unless other means are provided for readily and accurately determining the amount of water in the tank.

NOTE 11—The scale accuracy limitations of the National Ready Mixed Concrete Association Plant Certification meet the requirements of Specification C 94.

10. Mixers and Agitators

10.1 Mixers may be stationary mixers or truck mixers. Agitators may be truck mixers or truck agitators.

10.1.1 Stationary mixers shall be equipped with a metal plate or plates on which are plainly marked the mixing speed of the drum or paddles, and the maximum capacity in terms of the volume of mixed concrete. When used for the complete mixing of concrete, stationary mixers shall be equipped with an acceptable timing device that will not permit the batch to be discharged until the specified mixing time has elapsed.

10.1.2 Each truck mixer or agitator shall have attached thereto in a prominent place a metal plate or plates on which are plainly marked the gross volume of the drum, the capacity of the drum of container in terms of the volume of mixed concrete, and the minimum and maximum mixing speeds of rotation of the drum, blades, or paddles. When the concrete is truck mixed as described in 11.5, or shrink mixed as described in 11.4, the volume of mixed concrete shall not exceed 63 % of the total volume of the drum or container. When the concrete is central mixed as described in 11.3, the volume of concrete in the truck mixer or agitator shall not exceed 80 % of the total volume of the drum or container. Truck mixers and agitators shall be equipped with means by which the number of revolutions of the drum, blades, or paddles may be readily verified.

10.2 All stationary and truck mixers shall be capable of combining the ingredients of the concrete within the specified time or the number of revolutions specified in 10.5, into a thoroughly mixed and uniform mass and of discharging the concrete so that not less than 5 of the 6 requirements shown in Table A1.1 shall have been met.

NOTE 12—The sequence or method of charging the mixer will have an important effect on the uniformity of the concrete.

10.3 The agitator shall be capable of maintaining the mixed concrete in a thoroughly mixed and uniform mass and of discharging the concrete with a satisfactory degree of uniformity as defined by Annex A1.

10.4 Slump tests of individual samples taken after discharge of approximately 15 % and 85 % of the load may be made for a quick check of the probable degree of uniformity (Note 13). These two samples shall be obtained within an elapsed time of not more than 15 min. If these slumps differ more than that specified in Annex A1, the mixer or agitator shall not be used unless the condition is corrected, except as provided in 10.5.

NOTE 13—No samples should be taken before 10 % or after 90 % of the batch has been discharged. Due to the difficulty of determining the actual quantity of concrete discharged, the intent is to provide samples that are representative of widely separated portions, but not the beginning and end of the load.

10.5 Use of the equipment may be permitted when operation with a longer mixing time, a smaller load, or a more efficient charging sequence will permit the requirements of Annex A1 to be met.

10.6 Mixers and agitators shall be examined or weighed routinely as frequently as necessary to detect changes in condition due to accumulations of hardened concrete or mortar and examined to detect wear of blades. When such changes are extensive enough to affect the mixer performance, the proof-tests described in Annex A1 shall be performed to show whether the correction of deficiencies is required.

11. Mixing and Delivery

11.1 Ready-mixed concrete shall be mixed and delivered to the point designated by the purchaser by means of one of the following combinations of operations:

11.1.1 *Central-Mixed Concrete.*

11.1.2 *Shrink-Mixed Concrete.*

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11.1.3 *Truck-Mixed Concrete.*

11.2 Mixers and agitators shall be operated within the limits of capacity and speed of rotation designated by the manufacturer of the equipment.

11.3 *Central-Mixed Concrete*—Concrete that is mixed completely in a stationary mixer and transported to the point of delivery either in a truck agitator, or a truck mixer operating at agitating speed, or in nonagitating equipment approved by the purchaser and meeting the requirements of Section 12, shall conform to the following: The mixing time shall be counted from the time all the solid materials are in the drum. The batch shall be so charged into the mixer that some water will enter in advance of the cement and aggregate, and all water shall be in the drum by the end of the first one fourth of the specified mixing time.

11.3.1 Where no mixer performance tests are made, the acceptable mixing time for mixers having capacities of 1 yd^3 (0.76 m^3) or less shall be not less than 1 min. For mixers of greater capacity, this minimum shall be increased 15 s for each cubic yard or fraction thereof of additional capacity.

11.3.2 Where mixer performance tests have been made on given concrete mixtures in accordance with the testing program set forth in the following paragraphs, and the mixers have been charged to their rated capacity, the acceptable mixing time may be reduced for those particular circumstances to a point at which satisfactory mixing defined in 11.3.3 shall have been accomplished. When the mixing time is so reduced the maximum time of mixing shall not exceed this reduced time by more than 60 s for air-entrained concrete.

11.3.3 *Sampling for Uniformity Tests of Stationary Mixers*—Samples of concrete for comparative purposes shall be obtained immediately after arbitrarily designated mixing times, in accordance with one of the following procedures:

11.3.3.1 *Alternative Procedure 1*—The mixer shall be stopped, and the required samples removed by any suitable means from the concrete at approximately equal distances from the front and back of the drum, or

11.3.3.2 *Alternative Procedure 2*—As the mixer is being emptied, individual samples shall be taken after discharge of approximately 15 % and 85 % of the load. Any appropriate method of sampling may be used, provided the samples are representative of widely separated portions, but not the very ends of the batch (Note 13).

11.3.3.3 The samples of concrete shall be tested in accordance with Section 19, and differences in test results for the two samples shall not exceed those given in Annex A1. Mixer performance tests shall be repeated whenever the appearance of the concrete or the coarse aggregate content of samples selected as outlined in this section indicates that adequate mixing has not been accomplished.

11.4 *Shrink-Mixed Concrete*—Concrete that is first partially mixed in a stationary mixer, and then mixed completely in a truck mixer, shall conform to the following: The time of partial mixing shall be minimum required to intermingle the ingredients. After transfer to a truck mixer the amount of mixing at the designated mixing speed will be that necessary to meet the requirements for uniformity of concrete as indicated in Annex A1. Tests to confirm such performance may be made in accordance with 11.3.3 and 11.3.3.3. Additional turning of the mixer, if any, shall be at a designated agitating speed.

11.5 *Truck-Mixed Concrete*—Concrete that is completely mixed in a truck mixer, 70 to 100 revolutions at the mixing speed designated by the manufacturer to produce the uniformity of concrete indicated in Annex A1. Concrete uniformity tests may be made in accordance with 11.5.1 and if requirements for uniformity of concrete indicated in Annex A1 are not met with 100 revolutions of mixing, after all ingredients including water, are in the drum, that mixer shall not be used until the condition is corrected, except as provided in 10.5. When satisfactory performance is found in one truck mixer, the performance of mixers of substantially the same design and condition of blades may be regarded as satisfactory. Additional revolutions of the mixer beyond the number found to produce the required uniformity of concrete shall be at a designated agitating speed.

11.5.1 *Sampling for Uniformity of Concrete Produced in Truck Mixers*—The concrete shall be discharged at the normal operating rate for the mixer being tested, with care being exercised not to obstruct or retard the discharge by an incompletely opened gate or seal. Separate samples, each consisting of approximately 2 ft^3 (0.1 m^3 approximately) shall be taken after discharge of approximately 15 % and 85 % of the load (Note 13). These samples shall be obtained within an elapsed time of not more than 15 min. The samples shall be secured in accordance with Method C 172, but shall be kept separate to represent specific points in the batch rather than combined to form a composite sample. Between samples, where necessary to maintain slump, the mixer may be turned in mixing direction at agitating speed. During sampling the receptacle shall receive the full discharge of the chute. Sufficient personnel must be available to perform the required tests promptly. Segregation during sampling and handling must be avoided. Each sample shall be remixed the minimum amount to ensure uniformity before specimens are molded for a particular test.

11.6 When a truck mixer or truck agitator is used for transporting concrete that has been completely mixed in a stationary mixer, any turning during transportation shall be at the speed designated by the manufacturer of the equipment as agitating speed.

11.7 When a truck mixer or agitator is approved for mixing or delivery of concrete, no water from the truck water system or elsewhere shall be added after the initial introduction of mixing water for the batch except when on arrival at the job site the slump of the concrete is less than that specified. Such additional water to bring the slump within required limits shall be injected into the mixer under such pressure and direction of flow that the requirements for uniformity specified in Annex A1 are met. The drum or blades shall be turned an additional 30 revolutions or more if necessary, at mixing speed, until the uniformity of the concrete is within these limits. Water shall not be added to the batch at any later time. Discharge of the concrete shall be completed within 1½ h, or before the drum has revolved 300 revolutions, whichever comes first, after the introduction of the mixing water to the cement and aggregates or the introduction of the cement to the aggregates. These limitations may be waived by the purchaser if the concrete is of such slump after the 1½-h time or 300-revolution limit has been reached that it can be placed, without the addition of water, to the batch. In hot weather, or under conditions contributing to quick stiffening of the concrete, a time less than 1½ h may be specified by the purchaser.

11.8 Concrete delivered in cold weather shall have the applicable minimum temperature indicated in the following table. (The purchaser shall inform the producer as to the type of construction for which the concrete is intended.)

	Minimum Concrete Temperature	
Air Temperature	Thin Sections and Unformed Slabs	Heavy Sections and Mass Concrete
	°F	
30 to 45	60	50
0 to 30	65	55
Below 0	70	60
	°C	
−1 to 7	16	10
−18 to −1	18	13
Below −18	21	16

The maximum temperature of concrete produced with heated aggregates, heated water, or both, shall at no time during its production or transportation exceed 90°F (32°C).

NOTE 14—When hot water is used rapid stiffening may occur if hot water is brought in direct contact with the cement. Additional information on cold weather concreting is contained in ACI 306R.

11.9 The producer shall deliver the ready mixed concrete during hot weather at concrete temperatures as low as practicable, subject to the approval of the purchaser.

NOTE 15—In some situations difficulty may be encountered when concrete temperatures approach 90°F (32°C). Additional information may be found in the Bureau of Reclamation Concrete Manual and in ACI 305R.

12. Use of Nonagitating Equipment

12.1 Central-mixed concrete may be transported in suitable nonagitating equipment approved by the purchaser. The proportions of the concrete shall be approved by the purchaser and the following limitations shall apply:

12.2 Bodies of nonagitating equipment shall be smooth, watertight, metal containers equipped with gates that will permit control of the discharge of the concrete. Covers shall be provided for protection against the weather when required by the purchaser.

12.3 The concrete shall be delivered to the site of the work in a thoroughly mixed and uniform mass and discharged with a satisfactory degree of uniformity as prescribed in Annex A1.

12.4 Slump tests of individual samples taken after discharge of approximately 15 % and 85 % of the load may be made for a quick check of the probable degree of uniformity (Note 13). These two samples shall be obtained within an elapsed time of not more than 15 min. If these slumps differ more than that specified in Table A1.1, the nonagitating equipment shall not be used unless the conditions are corrected as provided in 12.5.

12.5 If the requirements of Annex A1 are not met when the nonagitating equipment is operated for the maximum time of haul, and with the concrete mixed the minimum time, the equipment may still be used when operated using shorter hauls, or longer mixing times, or combinations thereof that will result in the requirements of Annex A1 being met.

13. Inspection: Materials, Production, Delivery

13.1 The manufacturer shall afford the inspector all reasonable access, without charge, for making necessary checks of the production facilities and for securing necessary samples to determine if the concrete is being produced in accordance with this specification. All tests and inspection shall be so conducted as not to interfere unnecessarily with the manufacture and delivery of the concrete.

14. Inspection of Fresh Concrete and Sampling

14.1 The contractor shall afford the inspector all reasonable access, without charge, for the procurement of samples of fresh concrete at time of placement to determine conformance of it to this specification.

14.2 Samples of concrete shall be obtained in accordance with Method C 172, except when taken to determine uniformity of slump within any one batch or load of concrete (10.4, 11.3.3, 11.5.1, and 12.4).

14.3 The individual who samples and tests concrete to determine if the concrete is being produced in accordance with this specification shall have demonstrated a knowledge and ability to perform the necessary test procedures equivalent to the minimum guidelines for certification of Concrete Field Testing Technicians, Grade I in accordance with ACI CP-2 (Note 16).

NOTE 16—Personnel of laboratories accredited for Testing Freshly Mixed Field Concrete under the National Voluntary Laboratory Accreditation Program (NVLAP) should be considered equivalent to ACI Grade I.

14.4 When the strength of concrete is used as a basis for acceptance, the manufacturer shall be entitled to copies of all test reports.

15. Slump and Air Content

15.1 Slump and air-content tests shall be made at the time of placement at the option of the inspector as often as is necessary for control checks and acceptance purposes, and always when strength specimens are made (17.2).

15.2 If the measured slump or air content falls outside the specified limits, a check test shall be made immediately on another portion of the same sample. In the event of a second failure, the concrete shall be considered to have failed the requirements of the specification.

16. Batch Ticket Information

16.1 The manufacturer of the concrete shall furnish to the purchaser with each batch of concrete before unloading at the site, a delivery ticket on which is printed, stamped, or written, information concerning said concrete as follows:

16.1.1 Name of ready-mix batch plant,

16.1.2 Serial number of ticket,

16.1.3 Date,

16.1.4 Truck number,

16.1.5 Name of purchaser,

16.1.6 Specific designation of job (name and location),

16.1.7 Specific class or designation of the concrete in conformance with that employed in job specifications,

16.1.8 Amount of concrete in cubic yards (or cubic metres),

16.1.9 Time loaded or of first mixing of cement and aggregates, and

16.1.10 Water added by receiver of concrete and his initials.

16.2 Additional information for certification purposes as designated by the purchaser and required by the job specifications shall be furnished when requested; such information may include:

16.2.1 Reading of revolution counter at the first addition of water,

16.2.2 Type and brand, and amount of cement,

16.2.3 Type and brand, and amount of admixtures,

16.2.4 Information necessary to calculate the total mixing water added by the producer. Total mixing water includes free water on the aggregates, water, and ice batched at the plant, and water added by the truck operator from the mixer tank,

16.2.5 Maximum size of aggregate,

16.2.6 Weights of fine and coarse aggregate,

16.2.7 Ingredients certified as being previously approved, and

16.2.8 Signature or initials of ready-mix representative.

17. Strength

17.1 When strength is used as a basis for acceptance of concrete, standard specimens shall be made in accordance to Method C 31. The specimens shall be cured under standard moisture and temperature conditions in accordance with Sections 8.2 and 8.3 of Method C 31 (see Section 19).

17.2 Strength tests as well as slump and air content tests shall generally be made with a frequency of not less than one test for each 150 yd^3 (115 m^3). Each test shall be made from a separate batch. On each day concrete is delivered, at least one strength test shall be made for each class of concrete.

17.3 For a strength test, at least two standard test specimens shall be made from a composite sample secured as required in Section 14. A test shall be the average of the strengths of the specimens tested at the age specified in 6.3.1 or 6.4.1 (Note 17). If a specimen shows definite evidence other than low strength, of improper sampling, molding, handling, curing, or testing, it shall be discarded and the strength of the remaining cylinder shall then be considered the test result.

NOTE 17—Additional tests may be made at other ages to obtain information on the adequacy of the strength development or to check the adequacy of curing and protection of the concrete. Specimens made to check the adequacy of curing and protection should be cured in accordance with 8.4 of Method C 31.

17.4 The representative of the purchaser shall ascertain and record the delivery-ticket number for the concrete and the exact location in the work at which each load represented by a strength test is deposited.

17.5 To conform to the requirements of this specification, the average of all of the strength tests (see 17.3) representing each class of concrete shall be sufficient to ensure that the following requirements are met (Note 18 and Note 19).

17.5.1 For concrete in structures designed by the working stress method and all construction other than that covered in 17.5.2, not more than 20 % of the strength tests shall have values less than the specified strength, f'_c and the average of any six consecutive strength tests (Note 18) shall be equal to or greater than the specified strength.

17.5.2 For concrete, in structures designed by the ultimate strength method and in prestressed structures, not more than 10 % of the strength tests shall have values less than the specified strength, f'_c, and the average of any three consecutive strength tests (Note 19) shall be equal to or greater than the specified strength.

NOTE 18—Due to variations in materials, operations, and testing, the average strength necessary to meet these requirements will be substantially higher than the specified strength. The amount higher increases as these variations increase and decrease as they are reduced. This is a function of the coefficient of variation and other factors of control explained in ACI 214. Pertinent data will be found in Table 4.

NOTE 19—When the number of tests made of any class of concrete total six or less, the average of all the tests shall be equal to or greater than shown in the following table:

No. of Tests	Required Average Strength of Consecutive Tests, f'_c Section 17.5.1	Section 17.5.2
1	0.79	0.86
2	0.90	0.97
3	0.94	1.02
4	0.97	1.05
5	0.99	1.07
6	1.00	1.08

18. Failure to Meet Strength Requirements

18.1 In the event that concrete tested in accordance with the requirements of Section 17 fails to meet the strength requirements of this specification, the manufacturer of the ready-mixed concrete and the purchaser shall confer to determine whether agreement can be reached as to what adjustment, if any, shall be made. If an agreement on a mutually satisfactory adjustment cannot be reached by the manufacturer and the purchaser, a decision shall be made by a panel of three qualified engineers, one of whom shall be designated by the purchaser, one by the manufacturer, and the third chosen by these two members of the panel. The question of responsibility for the cost of such arbitration shall be determined by the panel. Its decision shall be binding, except as modified by a court decision.

19. Methods of Sampling and Testing

19.1 Test ready-mixed concrete in accordance with the following methods:

19.1.1 *Compression Test Specimens*—Method C 31, using standard moist curing in accordance with 9.2 and 9.3 of Method C 31.

19.1.2 *Compression Tests*—Test Method C 39.

19.1.3 *Yield, Weight per Cubic Foot*—Test Method C 138.

19.1.4 *Air Content*—Test Method C 138; Test Method C 173 or Test Method C 231.

19.1.5 *Slump*—Test Method C 143.

19.1.6 *Sampling Fresh Concrete*—Method C 172.

19.2 The testing laboratory performing acceptance tests of concrete shall meet the requirements of Recommended Practice E 329.

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TABLE 4 Strength Requirements

Average Strength Requirements for Limiting Probability of Tests Falling Below the Specified Strength, f'_c, to One Out of Every Ten Tests					
Coefficient of Variation	5	10	15	20	25
Required Overdesign Factor	1.07	1.15	1.24	1.34	1.47
Design Strength	Required Average Strength[A]				
2000 psi	2140	2300	2480	2680	2940
2500 psi	2675	2875	3100	3350	3675
3000 psi	3210	3450	3720	4030	4420
3500 psi	3745	4025	4340	4690	5145
4000 psi	4270	4590	4960	5380	5890
4500 psi	4815	5175	5580	6030	6615
5000 psi	5340	5740	6200	6720	7360

Average Strength Requirements for Limiting Probability of Tests Falling Below the Specified Strength, f'_c, to One Out of Every Five Tests					
Coefficient of Variation	5	10	15	20	25
Required Overdesign Factor	1.04	1.09	1.14	1.20	1.27
Design Strength	Required Average Strength[A]				
2000 psi	2080	2180	2280	2400	2450
2500 psi	2600	2725	2850	3000	3180
3000 psi	3120	3270	3420	3600	3810
3500 psi	3640	3820	3990	4200	4450
4000 psi	4160	4360	4560	4800	5080
4500 psi	4680	4910	5130	5400	5720
5000 psi	5200	5450	5700	6000	6350

[A] Computed from Eq 4-1 and values of "t" for more than 30 samples from Table 4.1 (ACI 214-77). In the absence of statistical experience a coefficient of variation of 20 % shall be assumed.

ANNEX

(Mandatory Information)

A1. CONCRETE UNIFORMITY REQUIREMENTS

A1.1 The variation within a batch as provided in Table A1.1 shall be determined for each property listed as the difference between the highest value and the lowest value obtained from the different portions of the same batch. For this specification the comparison will be between two samples, representing the first and last portions of the batch being tested. Test results conforming to the limits of five of the six tests listed in Table A1.1 shall indicate uniform concrete within the limits of this specification.

A1.2 *Coarse Aggregate Content,* using the washout test, shall be computed from the following relations:

$$P = (c/b) \times 100$$

where:

P = weight % of coarse aggregate in concrete,
c = saturated surface-dry-weight in lb (kg) of aggregate retained on the No. 4 (4.75-mm) sieve, resulting from washing all material finer than this sieve from the fresh concrete, and
b = weight of sample of fresh concrete in unit weight container, lb (kg).

A1.3 *Unit Weight of Air Free Mortar* shall be calculated as follows:

Inch-pound units:

$$M = \frac{b - c}{V - \left(\frac{V \times A}{100} + \frac{c}{G \times 62.4}\right)}$$

Metric units:

$$M = \frac{b - c}{V - \left(\frac{V \times A}{100} + \frac{c}{1000G}\right)}$$

where:

M = unit weight of air-free mortar, lb/ft^3 (kg/m^3),
b = weight of concrete sample in unit weight container, lb (kg),
c = saturated surface-dry-weight of aggregate retained on No. 4 (4.75-mm) sieve, lb (kg),
V = volume of unit weight container, ft^3 (m^3),
A = air content of concrete, %, measured in accordance with 18.1.4 on the sample being tested, and
G = specific gravity of coarse aggregate (SSD).

TABLE A1.1 Requirements for Uniformity of Concrete

Test	Requirement, Expressed as Maximum Permissible Difference in Results of Tests of Samples Taken from Two Locations in the Concrete Batch
Weight per cubic foot (weight per cubic metre) calculated to an air-free basis, lb/ft^3 (kg/m^3)	1.0 (16)
Air content, volume % of concrete	1.0
Slump:	
If average slump is 4 in. (102 mm) or less, in. (mm)	1.0 (25)
If average slump is 4 to 6 in. (102 to 152 mm), in. (mm)	1.5 (38)
Coarse aggregate content, portion by weight of each sample retained on No. 4 (4.75-mm) sieve, %	6.0
Unit weight of air-free mortar[A] based on average for all comparative samples tested, %.	1.6
Average compressive strength at 7 days for each sample,[B] based on average strength of all comparative test specimens, %	7.5[C]

[A] "Test for Variability of Constituents in Concrete," Designation 26, *Bureau of Reclamation Concrete Manual*, 7th Edition. Available from Superintendent of Documents, U. S. Government Printing Office, Washington, DC 20402.

[B] Not less than 3 cylinders will be molded and tested from each of the samples.

[C] Tentative approval of the mixer may be granted pending results of the 7-day compressive strength tests.

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This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, PA 19103.

American Association State Highway and Transportation Officials Standard AASHTO No.: T 121

Standard Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete[1]

This standard is issued under the fixed designation C 138; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

[ε1] NOTE—Editorial changes were made throughout in January 1983.

1. Scope

1.1 This method covers determination of the weight per cubic foot or cubic metre of freshly mixed concrete and gives formulas for calculating the yield, cement content, and the air content of the concrete. Yield is defined as the volume of concrete produced from a mixture of known quantities of the component materials.

1.2 The values stated in inch-pound units are to be regarded as the standard.

2. Referenced Documents

2.1 *ASTM Standards:*

C 29 Test Method for Unit Weight and Voids in Aggregate[2]

C 150 Specification for Portland Cement[2,3]

C 172 Method of Sampling Freshly Mixed Concrete[2]

C 188 Test Method for Density of Hydraulic Cement[3]

C 231 Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method[2]

3. Symbols

A = air content (percentage of voids) in the concrete

N = actual cement content, lb/yd^3 or kg/m^3

N_t = weight of cement in the batch, lb or kg

R_y = relative yield

T = theoretical weight of the concrete computed on an airfree basis, lb/ft^3 or kg/m^3 (Note 1)

V = total absolute volume of the component ingredients in the batch, ft^3 or m^3

W = unit weight of concrete, lb/ft^3 or kg/m^3

W_1 = total weight of all materials batched, lb or kg (Note 2)

Y = volume of concrete produced per batch, yd^3 or m^3

Y_d = volume of concrete which the batch was designed to produce, yd^3 (m^3)

Y_f = volume of concrete produced per batch, ft^3

NOTE 1—The theoretical weight per cubic foot or cubic metre is, customarily, a laboratory determination, the value for which is assumed to remain constant for all batches made using identical component ingredients and proportions. It is calculated from the equation.

$$T = W_1/V$$

The absolute volume of each ingredient in cubic feet is equal to the quotient of the weight of that ingredient divided by the product of its specific gravity times 62.4. The absolute volume of each ingredient in cubic metres is equal to the weight of the ingredient in kilograms divided by 1000 times its specific gravity. For the aggregate components, the bulk specific gravity and weight should be based on the saturated, surface-dry condition. For cement, the actual specific gravity should be determined by Test Method C 188. A value of 3.15 may be used for cements manufactured to meet the requirements of Specification C 150.

NOTE 2—The total weight of all materials batched is the sum of the weights of the cement, the fine aggregate in the condition used, the coarse aggregate in the condition used, the mixing water added to the batch and any other solid or liquid materials used.

4. Apparatus

4.1 *Balance*—A balance or scale accurate to within 0.3 % of the test load at any point within the range of use. The range of use shall be considered to extend from the weight of the measure empty to the weight of the measure plus its contents at 160 lb/ft^3 (2600 kg/m^3).

4.2 *Tamping Rod*—A round, straight steel rod, ⅝ in. (16 mm) in diameter and approximately 24 in. (600 mm) in length, having the tamping end rounded to a hemispherical tip the diameter of which is ⅝ in.

4.3 *Internal Vibrator*—Internal vibrators may have rigid or flexible shafts, preferably powered by electric motors. The frequency of vibration shall be 7000 vibrations per minute or greater while in use. The outside diameter or the side dimension of the vibrating element shall be at least 0.75 in. (19 mm) and not greater than 1.50 in. (38 mm). The length of the shaft shall be at least 24 in. (600 mm).

4.4 *Measure*—A cylindrical container made of steel or other suitable metal (Note 3). It shall be watertight and sufficiently rigid to retain its form and calibrated volume under rough usage. Measures that are machined to accurate dimensions on the inside and provided with handles are preferred. The minimum capacity of the measure shall conform to the requirements of Table 1. All measures, except for measuring bowls of air meters which are also used for Test Method C 138 tests, shall conform to the requirements of Test Method C 29. When measuring bowls of air meters are used, they shall conform to the requirements of Test Method C 231. The top rim of the air meter bowls shall be smooth and plane within 0.01 in. (0.25 mm).

[1] This method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.03 on Methods of Testing Fresh Concrete.

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[2] *Annual Book of ASTM Standards*, Vol 04.02.

[3] *Annual Book of ASTM Standards*, Vol 04.01.

TABLE 1 Minimun Capacity of Measures

Nominal Maximum Size of Coarse Aggregate[A]		Capacity of Measure, min[B]	
in.	mm	ft^3	dm^3
1	25.0	0.2	6
1½	37.5	0.4	11
2	50	0.5	14
3	75	1.0	28
4½	114	2.5	71
6	152	3.5	99

[A] Aggregate of a given nominal maximum size may contain up to 10 % of particles retained on the sieve referred to.

[B] To provide for wear, measures may be up to 5 % smaller than indicated in this table.

NOTE 3—The metal should not be readily subject to attack by cement paste. However, reactive materials such as aluminum alloys may be used in instances where as a consequence of an initial reaction, a surface film is rapidly formed which protects the metal against further corrosion.

NOTE 4—The top rim is satisfactorily plane if a 0.01-in. (0.25-mm) feeler gage cannot be inserted between the rim and a piece of ¼ in. (6 mm) or thicker plate glass laid over the top of the measure.

4.5 *Strike-Off Plate*—A flat rectangular metal plate at least ¼ in. (6 mm) thick or a glass or acrylic plate at least ½ in. (12 mm) thick with a length and width at least 2 in. (50 mm) greater than the diameter of the measure with which it is to be used. The edges of the plate shall be straight and smooth within a tolerance of 1/16 in. (1.5 mm).

4.6 *Calibration Equipment*—A piece of plate glass, preferably at least ¼ in. (6 mm) thick and at least 1 in. (25 mm) larger than the diameter of the measure to be calibrated. A supply of water pump or chassis grease that can be placed on the rim of the container to prevent leakage.

4.7 *Mallet*—A mallet (with a rubber or rawhide head) weighing approximately 1.25 ± 0.50 lb (0.57 ± 0.23 kg) for use with measures of 0.5 ft^3 (14 dm^3) or smaller, and a mallet weighing approximately 2.25 ± 0.50 lb (1.02 ± 0.23 kg) for use with measures larger than 0.5 ft^3.

5. Calibration of Measure

5.1 Calibrate the measure and determine the factor used to convert the weight in pounds or kilograms contained in the measure to weight in pounds per cubic foot or kilograms per cubic metre. Follow the procedure outlined in Test Method C 29. Measures shall be recalibrated at least once a year or whenever there is reason to question the accuracy of the calibration.

6. Sample

6.1 Obtain the sample of freshly mixed concrete in accordance with Method C 172.

7. Procedure

7.1 Compact measures smaller than 0.4 ft^3 (11 dm^3) by rodding because of the danger of excessive loss of entrained air. For measures 0.4 ft^3 or larger, base the selection of the method of consolidation on the slump, unless the method is stated in the specifications under which the work is being performed. The methods of consolidation are rodding and internal vibration. Rod concretes with a slump greater than 3 in. (75 mm). Rod or vibrate concrete with a slump of 1 to 3 in. (25 to 75 mm). Consolidate concretes with a slump less than 1 in. (25 mm) by vibration.

NOTE 5—The nonplastic concrete, such as is commonly used in the manufacture of pipe and unit masonry, is not covered by this method.

7.2 *Rodding*—Place the concrete in the measure in three layers of approximately equal volume. Rod each layer with 25 strokes of the tamping rod when the 0.5 ft^3 (14 dm^3) or smaller measures are used and 50 strokes when the 1 ft^3 (28 dm^3) measure is used. Rod the bottom layer throughout its depth but the rod shall not forcibly strike the bottom of the measure. Distribute the strokes uniformly over the cross section of the measure and for the top two layers, penetrate about 1 in. (25 mm) into the underlying layer. After each layer is rodded, tap the sides of the measure smartly 10 to 15 times with the appropriate mallet (see 4.7) to close any voids left by the tamping rod and to release any large bubbles of air that may have been trapped. Add the final layer so as to avoid overfilling.

7.3 *Internal Vibration*—Fill and vibrate the measure in two approximately equal layers. Place all of the concrete for each layer in the measure before starting vibration of that layer. Insert the vibrator at three different points for each layer. In compacting the bottom layer, do not allow the vibrator to rest on or touch the bottom or sides of the measure. In compacting the final layer, the vibrator shall penetrate into the underlying layer approximately 1 in. (25 mm). Take care that the vibrator is withdrawn in such a manner that no air pockets are left in the specimen. The duration of vibration required will depend upon the workability of the concrete and the effectiveness of the vibrator (Note 6). Continue vibration only long enough to achieve proper consolidation of the concrete (Note 7). Observe a constant duration of vibration for the particular kind of concrete, vibrator, and measure involved.

NOTE 6—Usually, sufficient vibration has been applied as soon as the surface of the concrete becomes relatively smooth.

NOTE 7—Overvibration may cause segregation and loss of appreciable quantities of intentionally entrained air.

7.4 On completion of consolidation the measure must not contain a substantial excess or deficiency of concrete. An excess of concrete protruding approximately ⅛ in. (3 mm) above the top of the mold is optimum. A small quantity of concrete may be added to correct a deficiency. If the measure contains a great excess of concrete at completion of consolidation, remove a representative portion of the excess concrete with a trowel or scoop immediately following completion of consolidation and before the measure is struck-off.

7.5 *Strike-Off*—After consolidation, strike-off the top surface of the concrete and finish it smoothly with the flat strike-off plate using great care to leave the measure just level full. The strike-off is best accomplished by pressing the strike-off plate on the top surface of the measure to cover about two thirds of the surface and withdrawing the plate with a sawing motion to finish only the area originally covered. Then place the plate on the top of the measure to cover the original two thirds of the surface and advance it with a vertical pressure and a sawing motion to cover the whole surface of the measure. Several final strokes with the inclined edge of the plate will produce a smooth finished surface.

7.6 *Cleaning and Weighing*—After strike-off, clean all excess concrete from the exterior of the measure and determine the net weight of the concrete in the measure to an accuracy consistent with the requirements of 4.1.

8. Calculations

8.1 *Unit Weight*—Calculate the net weight of the concrete in pounds or kilograms by subtracting the weight of the measure from the gross weight. Calculate the unit weight, W, by multiplying the net weight by the calibration factor for the measure used, determined according to Test Method C 29.

8.2 *Yield*—Calculate the yield as follows:

$$Y_f(\text{ft}^3) = W_1/W$$

or,

$$Y(\text{yd}^3) = W_1/(27\ W)$$

or,

$$Y(\text{m}^3) = W_1/W$$

8.3 *Relative Yield*—Relative yield is the ratio of the actual volume of concrete obtained to the volume as designed for the batch calculated as follows:

$$R_y = Y/Y_d$$

NOTE 8—A value for R_y greater than 1.00 indicates an excess of concrete being produced whereas a value less than this indicates the batch to be "short" of its designed volume.

8.4 *Cement Content*—Calculate the actual cement content as follows:

$$N = N_t/Y$$

8.5 *Air Content*—Calculate the air content as follows:

$$A = [(T - W)/T] \times 100$$

or,

$$A = [(Y_f - V)/Y_f] \times 100 \text{ (Inch-pound units)}$$

or,

$$A = [(Y - V)/Y] \times 100 \text{ (SI units)}$$

9. Precision

9.1 Data are being compiled and developed that will be suitable for use in developing precision statements for this test method.

Standard Test Method for Slump of Portland Cement Concrete[1]

This standard is issued under the fixed designation C 143; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This method has been approved for use by agencies of the Department of Defense and for listing in the DoD Index of Specifications and Standards.

ε1 Note—Section 2 was added editorially and subsequent sections renumbered in August 1985.

1. Scope

1.1 This test method covers determination of slump of concrete, both in the laboratory and in the field.

Note 1—This test method is considered applicable to plastic concrete having coarse aggregate up to 1½ in. (38 mm) in size. If the coarse aggregate is larger than 1½ in. in size, the method is applicable when it is made on the fraction of concrete passing a 1½-in. sieve with the larger aggregate being removed in accordance with Section 4 of Method C 172. This test method is not considered applicable to nonplastic and noncohesive concrete.

1.2 The values stated in inch-pound units are to be regarded as the standard. The metric equivalents of inch-pound units may be approximate.

2. Referenced Document

2.1 *ASTM Standard:*

C 172 Method of Sampling Freshly Mixed Concrete[2]

3. Apparatus

3.1 *Mold*—The test specimen shall be formed in a mold made of metal not readily attacked by the cement paste. The metal shall not be thinner than No. 16 gage (BWG) and if formed by the spinning process, there shall be no point on the mold at which the thickness is less than 0.045 in. (1.14 mm). The mold shall be in the form of the lateral surface of the frustum of a cone with the base 8 in. (203 mm) in diameter, the top 4 in. (102 mm) in diameter, and the height 12 in. (305 mm). Individual diameters and heights shall be within ±⅛ in. (3.2 mm) of the prescribed dimensions. The base and the top shall be open and parallel to each other and at right angles to the axis of the cone. The mold shall be provided with foot pieces and handles similar to those shown in Fig. 1. The mold may be constructed either with or without a seam. When a seam is required, it should be essentially as shown in Fig. 1. The interior of the mold shall be relatively smooth and free from projections such as protruding rivets. The mold shall be free from dents. A mold which clamps to a nonabsorbent base plate is acceptable instead of the one illustrated provided the clamping arrangement is such that it can be fully released without movement of the mold.

3.2 *Tamping Rod*—The tamping rod shall be a round, straight steel rod ⅝ in. (16 mm) in diameter and approximately 24 in. (600 mm) in length, having the tamping end rounded to a hemispherical tip the diameter of which is ⅝ in.

4. Sample

4.1 The sample of concrete from which test specimens are made shall be representative of the entire batch. It shall be obtained in accordance with Method C 172.

5. Procedure

5.1 Dampen the mold and place it on a flat, moist, nonabsorbent (rigid) surface. It shall be held firmly in place during filling by the operator standing on the two foot pieces. From the sample of concrete obtained in accordance with Section 4, immediately fill the mold in three layers, each approximately one third the volume of the mold.

Note 2—One third of the volume of the slump mold fills it to a depth of 2⅝ in. (67 mm); two thirds of the volume fills it to a depth of 6⅛ in. (155 mm).

5.2 Rod each layer with 25 strokes of the tamping rod. Uniformly distribute the strokes over the cross section of each layer. For the bottom layer this will necessitate inclining the rod slightly and making approximately half of the strokes near the perimeter, and then progressing with vertical strokes spirally toward the center. Rod the bottom layer throughout its depth. Rod the second layer and the top layer each throughout its depth, so that the strokes just penetrate into the underlying layer.

5.3 In filling and rodding the top layer, heap the concrete above the mold before rodding is started. If the rodding

[1] This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.03 on Methods of Testing Fresh Concrete.

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[2] *Annual Book of ASTM Standards*, Vol 04.02.

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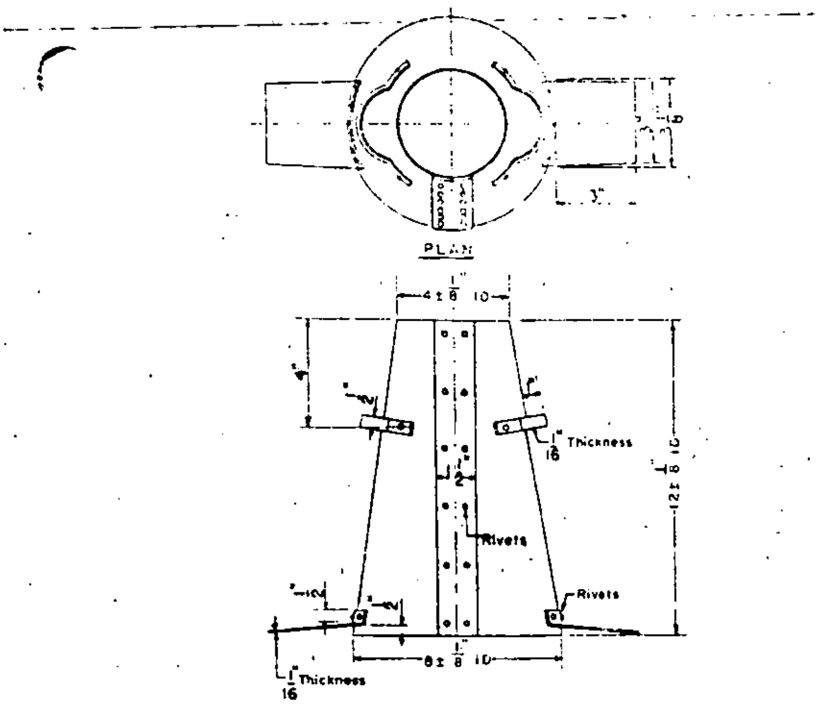

Metric Equivalents

in.	1/16	1/8	1/2	1	1 1/2	3	3 1/8	4	8	12
mm	1.6	3.2	12.7	25.4	38.1	76.2	79.4	102	203	305

FIG. 1 Mold for Slump Test

operation results in subsidence of the concrete below the top edge of the mold, add additional concrete to keep an excess of concrete above the top of the mold at all times. After the top layer has been rodded, strike off the surface of the concrete by means of a screeding and rolling motion of the tamping rod. Remove the mold immediately from the concrete by raising it carefully in a vertical direction. Raise the mold a distance of 12 in. (300 mm) in 5 ± 2 s by a steady upward lift with no lateral or torsional motion. Complete the entire test from the start of the filling through removal of the mold without interruption and complete it within an elapsed time of 2½ min.

5.4 Immediately measure the slump by determining the vertical difference between the top of the mold and the displaced original center of the top surface of the specimen. If a decided falling away or shearing off of concrete from one side or portion of the mass occurs (Note 3), disregard the test and make a new test on another portion of the sample.

NOTE 3—If two consecutive tests on a sample of concrete show a falling away or shearing off of a portion of the concrete from the mass of the specimen, the concrete probably lacks necessary plasticity and cohesiveness for the slump test to be applicable.

6. Report

6.1 Record the slump in terms of inches (millimetres) to the nearest ¼ in. (6 mm) of subsidence of the specimen during the test as follows:

Slump = 12 − inches of height after subsidence

7. Precision and Bias

7.1 Data are being compiled and developed that will be suitable for use in developing precision statements for this test method.

The American Society for Testing and Materials takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this standard. Users of this standard are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.

Designation: C 172 – 82

Standard Method of Sampling Freshly Mixed Concrete[1]

This standard is issued under the fixed designation C 172; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This method has been approved for use by agencies of the Department of Defense and for listing in the DoD Index of Specifications and Standards.

1. Scope

1.1 This method covers procedures for obtaining representative samples of fresh concrete as delivered to the project site on which tests are to be performed to determine compliance with quality requirements of the specifications under which the concrete is furnished (Note 1). The method includes sampling from stationary, paving and truck mixers, and from agitating and nonagitating equipment used to transport central-mixed concrete.

1.2 The values stated in inch-pound units are to be regarded as the standard. The metric equivalents of inch-pound units may be approximate.

NOTE 1—Composite samples are required by this method, unless specifically excepted by procedures governing the tests to be performed such as tests to determine uniformity of consistency and mixer efficiency. Procedures used to select the specific test batches are not described in this method, but it is recommended that random sampling be used to determine over-all specification compliance.

1.3 This method also covers the procedures to be used for preparing a sample of concrete for further testing where it is desirable or necessary to remove the aggregate larger than a designated size. This removal of larger aggregate particles is preferably accomplished by wet-sieving.

2. Referenced Document

2.1 *ASTM Standard:*

E 11 Specification for Wire-Cloth Sieves for Testing Purposes[2]

3. Sampling

3.1 The elapsed time between obtaining the first and final portions of the composite sample shall be as short as possible, but in no instance shall it exceed 15 min.

3.1.1 Transport the individual samples to the place where fresh concrete tests are to be performed or where test specimens are to be molded. They shall be combined and remixed with a shovel the minimum amount necessary to ensure uniformity and compliance with the minimum time limits specified in 3.1.2.

3.1.2 Start tests for slump or air content, or both, within 5 min after obtaining the final portion of the composite sample. Complete these tests as expeditiously as possible. Start molding specimens for strength tests within 15 min after fabricating the composite sample. Keep the elapsed time between obtaining and using the sample as short as possible and protect the sample from the sun, wind, and other sources of rapid evaporation, and from contamination.

4. Procedure

4.1 *Size of Sample*—Make the samples to be used for strength tests a minimum of 1 ft^3 (28 L). Smaller samples may be permitted for routine air content and slump tests and the size shall be dictated by the maximum aggregate size.

4.2 The procedures used in sampling shall include the use of every precaution that will assist in obtaining samples that are truly representative of the nature and condition of concrete sampled as follows:

NOTE 2—Sampling should normally be performed as the concrete is delivered from the mixer to the conveying vehicle used to transport the concrete to the forms; however, specifications may require other points of sampling, such as the discharge of a concrete pump.

4.2.1 *Sampling from Stationary Mixers, Except Paving Mixers*—Sample the concrete at two or more regularly spaced intervals during discharge of the middle portion of the batch. Take the samples, so obtained, within the time limit specified in Section 3, and composite them into one sample for test purposes. Do not obtain samples from the very first or last portions of the batch discharge. Perform sampling by passing a receptacle completely through the discharge stream, or by completely diverting the discharge into a sample container. If discharge of the concrete is too rapid to divert the complete discharge stream, discharge the concrete into a container or transportation unit sufficiently large to accommodate the entire batch and then accomplish the sampling in the same manner as given above. Take care not to restrict the flow of concrete from the mixer, container, or transportation unit so as to cause segregation. These requirements apply to both tilting and nontilting mixers.

4.2.2 *Sampling from Paving Mixers*—Sample the concrete after the contents of the paving mixer have been discharged. Obtain samples from at least five different portions of the pile and then composite into one sample for test purposes. Avoid contamination with subgrade material or prolonged contact with and absorptive subgrade. To preclude contamination or absorption by the subgrade, sample the concrete by placing three shallow containers on the subgrade and discharging the concrete across the container. Composite the samples so obtained into one sample for test purposes. The containers shall be of a size sufficient to provide a composite sample size that is in agreement with the maximum aggregate size.

[1] This method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.03 on Methods of Testing Fresh Concrete.

Current edition approved May 28, 1982. Published July 1982. Originally published as C 172 – 42. Last previous edition C 172 – 71 (1977).

[2] *Annual Book of ASTM Standards*, Vols 04.02 and 14.02.

NOTE 3—In some instances, the containers may have to be supported above the subgrade to prevent displacement during discharge.

4.2.3 *Sampling from Revolving Drum Truck Mixers or Agitators*—Sample the concrete at two or more regularly spaced intervals during discharge of the middle portion of the batch. Take the samples so obtained within the time limit specified in Section 3 and composite them into one sample for test purposes. In any case do not obtain samples until after all of the water has been added to the mixer: also do not obtain samples from the very first or last portions of the batch discharge. Sample by repeatedly passing a receptacle through the entire discharge stream or by completely diverting the discharge into a sample container. Regulate the rate of discharge of the batch by the rate of revolution of the drum and not by the size of the gate opening.

4.2.4 *Sampling from Open-Top Truck Mixers, Agitators, Nonagitating Equipment, or Other Types of Open-Top Containers*—Take samples by whichever of the procedures described in 4.2.1, 4.2.2, or 4.2.3 is most applicable under the given conditions.

5. Additional Procedure for Large Maximum Size Aggregate Concrete

5.1 When the concrete contains aggregate larger than that appropriate for the size of the molds or equipment to be used, wet-sieve the sample as described below except make unit-weight tests for use in yield computations on the full mix.

NOTE 4—The effect of wet-sieving on the test results should be considered. For example, wet-sieving concrete causes the loss of a small amount of air due to additional handling. The air content of the wet-sieved fraction of concrete is greater than that of the total concrete because the larger size aggregate which is removed does not contain air. The apparent strength of wet-sieved concrete in smaller specimens is usually greater than that of the total concrete in larger appropriate size specimens. The effect of these differences may need to be considered or determined by supplementary testing for quality control or test result evaluation purposes.

5.2 *Definition*:

5.2.1 *wet-sieving concrete*—the process of removing aggregate larger than a designated size from the fresh concrete by sieving it on a sieve of the designated size.

5.3 *Apparatus*:

5.3.1 *Sieves*, as designated, conforming to Specification E 11.

5.3.2 *Wet-Sieving Equipment*—Equipment for wet-sieving concrete shall be a sieve as noted in 5.3.1 of suitable size and conveniently arranged and supported so that one can shake it rapidly by either hand or mechanical means. Generally, a horizontal back and forth motion is preferred. The equipment shall be capable of rapidly and effectively removing the designated size of aggregate.

5.3.3 *Hand Tools*—Shovels, hand scoops, plastering trowels, and rubber gloves as required.

5.4 *Procedure*:

5.4.1 *Wet-Sieving*—After sampling the concrete, pass the concrete over the designated sieve and remove and discard the aggregate retained. This shall be done before remixing. Shake or vibrate the sieve by hand or mechanical means until no undersize material remains on the sieve. Mortar adhering to the aggregate retained on the sieve shall not be wiped from it before it is discarded. Place only enough concrete on the sieve at any one time so that after sieving, the thickness of the layer of retained aggregate is not more than one particle thick. The concrete which passes the sieve shall fall into a batch pan of suitable size which has been dampened before use or onto a clean, moist, nonabsorbent surface. Scrape any mortar adhering to the sides of the wet-sieving equipment into the batch. After removing the larger aggregate particles by wet-sieving remix the batch with a shovel the minimum amount necessary to ensure uniformity and proceed testing immediately.

Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method[1]

This standard is issued under the fixed designation C 173; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This method has been approved for use by agencies of the Department of Defense and for listing in the DoD Index of Specifications and Standards.

1. Scope

1.1 This test method covers determination of the air content of freshly mixed concrete containing any type of aggregate, whether it be dense, cellular, or lightweight.

2. Referenced Documents

2.1 *ASTM Standards:*

C 29 Test Method for Unit Weight and Voids in Aggregate[2]

C 138 Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete[2]

C 172 Method of Sampling Freshly Mixed Concrete[2]

C 231 Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method[2]

3. Apparatus

3.1 *Airmeter*—An airmeter consisting of a bowl and a top section (Fig. 1) conforming to the following requirements:

3.1.1 *Bowl*—The bowl shall be constructed of machined metal of such thickness as to be sufficiently rigid to withstand normal field use and of such composition as not to be readily attacked by cement paste. The bowl shall have a diameter equal to 1 to 1.25 times the height and be constructed with a flange at or near the top surface. Bowls shall not have a capacity of less than 0.075 ft^3 (0.02 m^3).

3.1.2 *Top Section*—The top section shall be constructed of machined metal of thickness sufficiently rigid to withstand normal field use and of composition not readily attacked by cement paste. The top section shall have a capacity at least 20 % larger than the bowl and shall be equipped with a flexible gasket and with hooks or lugs to attach to the flange on the bowl to make a watertight connection. The top section shall be equipped with a glass-lined or transparent plastic neck, graduated in increments not greater than 0.5 % from 0 at the top to 9 %, or more, of the volume of the bowl. Graduations shall be accurate to ±0.1 % by volume of the bowl. The upper end of the neck shall be threaded and equipped with a screw cap having a gasket to make a watertight fit.

3.2 *Funnel*—A metal funnel with a spout of a size permitting it to be inserted through the neck of the top section and long enough to extend to a point just above the bottom of the top section. The discharge end of the spout shall be so constructed that when water is added to the container there will be a minimum disturbance of the concrete.

3.3 *Tamping Rod*—A round, straight steel rod, ⅝ in. or 16 mm in diameter at least 12 in. or 300 mm long with both ends rounded to a hemispherical tip of the same diameter.

3.4 *Strike-off Bar*—A flat, straight steel bar at least ⅛ by ¾ by 12 in. or 3 by 20 by 300 mm long.

3.5 *Measuring Cup*—A metal cup having a capacity equal to 1.03 ± 0.04 % of the volume of the bowl of the air meter.

NOTE 1—The volume of the measuring cup is slightly larger than 1.0 % of the volume of the bowl to compensate for the volume contraction that takes place when 70 % isopropyl alcohol is mixed with water. Other alcohols or defoaming agents may be used if calculations show that their use will result in an error in indicated air content less than 0.1 %.

3.6 *Syringe*—A small rubber bulb syringe having a capacity at least that of the measuring cup.

3.7 *Pouring Vessel*—A metal or glass container of approximately 1-qt or 1-L capacity.

3.8 *Trowel*—A blunt-nosed brick mason's trowel.

3.9 *Scoop*—A small metal scoop.

3.10 *Isopropyl Alcohol*—Use 70 % by volume isopropyl alcohol (approximately 65 % by weight). (Notes 1 and 2).

NOTE 2—Seventy % isopropyl alcohol is commonly available as rubbing alcohol. More concentrated grades can be diluted with water to the required concentration.

3.11 *Mallet*—A mallet (with a rubber or rawhide head) weighing approximately 1.25 ± 0.50 lb (0.57 ± 0.23 kg) for use with measures of 0.5 ft^3 (14 dm^3) or smaller, and a mallet weighing approximately 2.25 ± 0.50 lb (1.02 ± 0.23 kg) for use with measures larger than 0.5 ft^3.

4. Calibration

4.1 The volume of the bowl of the airmeter, in cubic feet or cubic metres shall be determined by accurately weighing the amount of water required to fill it at room temperature, and dividing this weight by the unit weight of water at the same temperature. Follow the calibration procedure outlined in Section 7 of Test Method C 29.

4.2 Determine the accuracy of the graduations on the neck of the top section of the airmeter by filling the assembled measuring bowl and top section with water to the level of the mark for any air content. Add a quantity of water

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[1] This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.03 on Methods of Testing Fresh Concrete.

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[2] *Annual Book of ASTM Standards*, Vol 04.02.

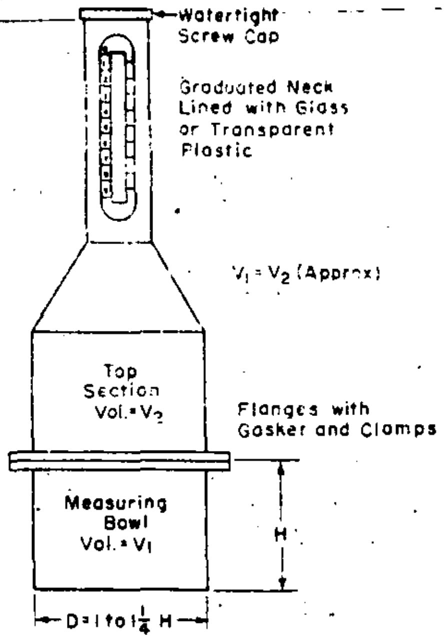

FIG. 1 Apparatus for Measuring Air Content of Fresh Concrete by Volumetric Method

at room temperature, equal to 1.0 % of the volume of the bowl, to the water already in the neck. The height of the water column shall increase by an amount equivalent to 1.0 % of air.

4.3 Determine the volume of the measuring cup using water at 70°F (21.1°C) by the method outlined in 4.1. A quick check can be made by adding one or more cups of water to the assembled apparatus and observing the increase in the height of the water column after filling to a given level as described in 4.2.

5. Sample

5.1 Obtain the sample of freshly mixed concrete in accordance with applicable provisions of Method C 172. If the concrete contains coarse aggregate particles that would be retained on a 1½-in. (37.5-mm) sieve, wet sieve a representative sample over a 1-in. (25-mm) sieve to yield somewhat more than enough material to fill the measuring bowl. The wet sieving procedure is described in Method C 172. Carry out the wet sieving operation with the minimum practicable disturbance of the mortar. Make no attempt to wipe adhering mortar from coarse aggregate particles retained on the sieve.

6. Procedure

6.1 *Rodding and Tapping*—Using the scoop, aided by the trowel if necessary, fill the bowl with freshly mixed concrete in three layers of equal depth. Rod each layer 25 times with the tamping rod. After each layer is rodded, tap the sides of the measure 10 to 15 times smartly with the mallet to close any voids left by the tamping rod and to release any large bubbles of air that may have been trapped.

6.2 *Striking Off*—After placement of the third layer of concrete in accordance with 6.1, strike off the excess concrete with the strike-off bar until the surface is flush with the top of the bowl. Wipe the flange of the bowl clean.

6.3 *Adding Water*—Clamp the top section into position on the bowl, insert the funnel, and add water until it appears in the neck. Remove the funnel and adjust the water level, using the rubber syringe, until the bottom of the meniscus is level with the zero mark. Attach and tighten the screw cap.

6.4 *Agitating and Rolling*—Invert and agitate the unit until the concrete settles free from the base; and then, with the neck elevated, roll and rock the unit until the air appears to have been removed from the concrete. Set the apparatus upright, jar it lightly, and allow it to stand until the air rises to the top. Repeat the operation until no further drop in the water column is observed.

6.5 *Dispelling Bubbles*—When all the air has been removed from the concrete and allowed to rise to the top of the apparatus, remove the screw cap. Add, in 1-cup increments using the syringe, sufficient isopropyl alcohol to dispel the foamy mass on the surface of the water.

6.6 *Reading*—Make a direct reading of the liquid in the neck, reading to the bottom of the meniscus, and estimating to the nearest 0.1 %.

7. Calculation

7.1 Calculate the air content percent of the concrete in the measuring bowl in percent by adding to the reading from 6.6 the amount of alcohol used in accordance with 6.5.

7.2 When the sample tested represents that portion of the mixture obtained by wet sieving over a 1-in. (25-mm) sieve, calculate the air content of the mortar or of the full mixture using the formulas given in Test Method C 231. Use appropriate quantities coarser or finer than the 1-in. sieve instead of the 1½-in. (37.5-mm) sieve specified in Test Method C 231.

8. Precision

8.1 Data are being compiled and developed that will be suitable for use in developing precision statements for this test method.

Designation: C 231 – 82ε2

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If not listed in the current combined index, will appear in the next edition.

Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method[1]

This standard is issued under the fixed designation C 231; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This method has been approved for use by agencies of the Department of Defense and for listing in the DoD Index of Specifications and Standards.

ε1 NOTE—Editorial changes were made throughout in January 1983.
ε2 NOTE—Section 5.2 was corrected editorially in June 1988.

1. Scope

1.1 This method covers determination of the air content of freshly mixed concrete from observation of the change in volume of concrete with a change in pressure.

1.2 This method is intended for use with concretes and mortars made with relatively dense aggregates for which the aggregate correction factor can be satisfactorily determined by the technique described in Section 5. It is not applicable to concretes made with lightweight aggregates, air-cooled blast-furnace slag, or aggregates of high porosity. In these cases, Test Method C 173, should be used.

1.3 The values stated in inch-pound units are to be regarded as the standard.

2. Referenced Documents

2.1 *ASTM Standards:*

C 138 Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete[2]

C 143 Test Method for Slump of Portland Cement Concrete[2]

C 172 Method of Sampling Freshly Mixed Concrete[2]

C 173 Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method[2]

C 192 Method of Making and Curing Concrete Test Specimens in the Laboratory[2]

3. Apparatus

3.1 *Air Meters*—There are available satisfactory apparatus of two basic operational designs employing the principle of Boyle's law. For purposes of reference herein these are designated Meter Type A and Meter Type B.

3.1.1 *Meter Type A*—An air meter consisting of a measuring bowl and cover assembly (see Fig. 1) conforming to the requirements of 3.2 and 3.3. The operational principle of this meter consists of introducing water to a predetermined height above a sample of concrete of known volume, and the application of a predetermined air pressure over the water. The determination consists of the reduction in volume of the air in the concrete sample by observing the amount the water level is lowered under the applied pressure, the latter amount being calibrated in terms of percent of air in the concrete sample.

3.1.2 *Meter Type B*—An air meter consisting of a measuring bowl and cover assembly (see Fig. 2) conforming to the requirements of 3.2 and 3.3. The operational principle of this meter consists of equalizing a known volume of air at a known pressure in a sealed air chamber with the unknown volume of air in the concrete sample, the dial on the pressure gage being calibrated in terms of percent air for the observed pressure at which equalization takes place. Working pressures of 7.5 to 30.0 psi (51 to 207 kPa) have been used satisfactorily.

3.2 *Measuring Bowl*—The measuring bowl shall be essentially cylindrical in shape, made of steel or other hard metal not readily attacked by the cement paste, having a minimum diameter equal to 0.75 to 1.25 times the height, and a capacity of at least 0.20 ft^3 (0.006 m^3). It shall be flanged or otherwise constructed to provide for a pressure tight fit between bowl and cover assembly. The interior surfaces of the bowl and surfaces of rims, flanges and other component fitted parts shall be machined smooth. The measuring bowl and cover assembly shall be sufficiently rigid to limit the expansion factor, *D*, of the apparatus assembly (Appendix X5) to not more than 0.1 % of air content on the indicator scale when under normal operating pressure.

3.3 *Cover Assembly:*

3.3.1 The cover assembly shall be made of steel or other hard metal not readily attacked by the cement paste. It shall be flanged or otherwise constructed to provide for a pressure tight fit between bowl and cover assembly and shall have machined smooth interior surfaces contoured to provide an air space above the level of the top of the measuring bowl. The cover shall be sufficiently rigid to limit the expansion factor of the apparatus assembly as prescribed in 3.2.

3.3.2 The cover assembly shall be fitted with a means of direct reading of the air content. The cover for the Type A meter shall be fitted with a standpipe, which may be a transparent graduated tube or may be a metal tube of uniform bore with a glass water gage attached. In the Type B meter, the dial of the pressure gage shall be calibrated to indicate the percent of air. Graduations shall be provided for

[1] This method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates, and is the direct responsibility of Subcommittee C09.03.03 on Methods of Testing Fresh Concrete.

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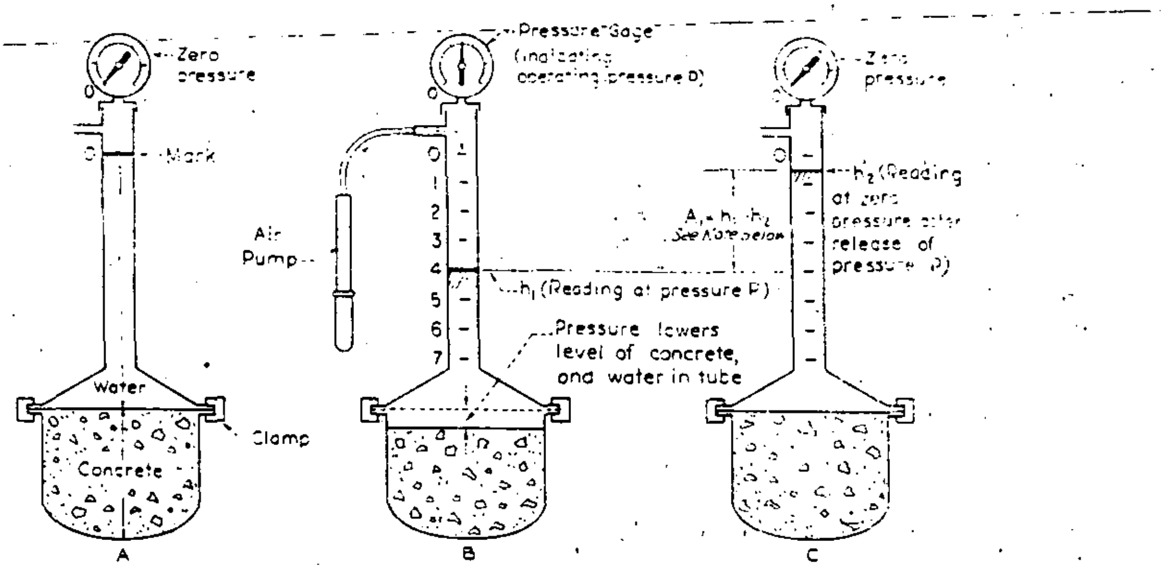
[2] *Annual Book of ASTM Standards*, Vol 04.02.



Note: $A_1 = h_1 - h_2$ when bowl contains concrete as shown in this figure; when bowl contains only aggregate and water, $h_1 - h_2 = G$ (aggregate correction factor). $A_1 - G = A$ (entrained air content of concrete)

FIG. 1 Illustration of the Pressure Method for Air Content—Type-A Meter

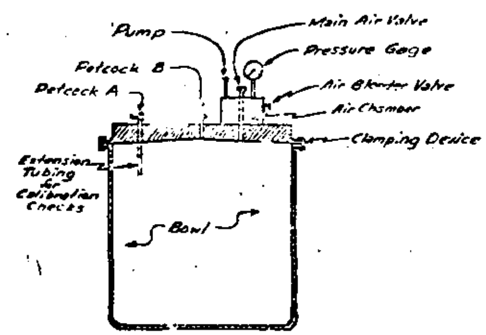


FIG. 2 Schematic Diagram—Type-B Meter

a range in air content of at least 8 % easily readable to 0.1 % as determined by the proper air pressure calibration test.

3.3.3 The cover assembly shall be fitted with air valves, air bleeder valves, and petcocks for bleeding off or through which water may be introduced as necessary for the particular meter design. Suitable means for clamping the cover to the bowl shall be provided to make a pressure-tight seal without entrapping air at the joint between the flanges of the cover and bowl. A suitable hand pump shall be provided with the cover either as an attachment or as an accessory.

3.4 *Calibration Vessel*—A measure having an internal volume equal to a percent of the volume of the measuring bowl corresponding to the approximate percent of air in the concrete to be tested; or, if smaller, it shall be possible to check calibration of the meter indicator at the approximate percent of air in the concrete to be tested by repeated filling of the measure. When the design of the meter requires placing the calibration vessel within the measuring bowl to check calibration, the measure shall be cylindrical in shape and of an inside depth ½ in. (13 mm) less than that of the bowl. A satisfactory measure of this type may be machined from No. 16 gage brass tubing, of a diameter to provide the volume desired, to which a brass disk ½ in. in thickness is soldered to form an end. When design of the meter requires withdrawing of water from the water-filled bowl and cover assembly to check calibration, the measure may be an integral part of the cover assembly or may be a separate cylindrical measure similar to the above described cylinder.

3.5 The designs of various available types of airmeters are such that they differ in operating techniques and therefore, all of the items described in 3.6 through 3.14 may not be required. The items required shall be those necessary for use with the particular design of apparatus used to satisfactorily determine air content in accordance with the procedures prescribed herein.

3.6 *Coil Spring or Other Device for Holding Calibration Cylinder in Place.*

3.7 *Spray Tube*—A brass tube of appropriate diameter, which may be an integral part of the cover assembly or which may be provided separately. It shall be so constructed that when water is added to the container, it is sprayed to the walls of the cover in such a manner as to flow down the sides causing a minimum of disturbance to the concrete.

3.8 *Trowel*—A standard brick mason's trowel.

3.9 *Tamping Rod*, as described in Method C 143.

3.10 *Mallet*—A mallet (with a rubber or rawhide head) weighing approximately 1.25 ± 0.50 lb (0.57 ± 0.23 kg) for use with measures of 0.5 ft^3 (14 dm^3) or smaller, and a mallet weighing approximately 2.25 ± 0.50 lb (1.02 ± 0.23 kg) for use with measures larger than 0.5 ft^3.

3.11 *Strike-Off Bar*—A flat straight bar of steel or other suitable metal.

3.12 *Funnel*, with the spout fitting into spray tube.

3.13 *Measure for Water*, having the necessary capacity to fill the indicator with water from the top of the concrete to the zero mark.

3.14 *Vibrator*, as described in Method C 192.

3.15 *Sieves*, 1½-in. (37.5-mm) with not less than 2 ft^2 (0.19 m^2) of sieving area.

4. Calibration of Apparatus

4.1 Make calibration tests in accordance with procedures prescribed in the appendix. Rough handling will affect the calibration of both Types A and B meters. Changes in barometric pressure will affect the calibration of Type A meter but not Type B meter. The steps described X1.2 to X1.6, as applicable to the meter type under consideration, are prerequisites for the final calibration test to determine the operating pressure, P, on the pressure gage of the Type A meter as described in X1.7, or to determine the accuracy of the graduations indicating air content on the dial face of the pressure gage of the Type B meter. Normally the steps in X1.2 to X1.6 need be made only once (at the time of initial calibration), or only occasionally to check volume constancy of the calibration cylinder and measuring bowl. On the other hand, the calibration test described in X1.7 and X1.9, as applicable to the meter type being checked, must be made as frequently as necessary to ensure that the proper gage pressure, P, is being used for the Type A meter or that the correct air contents are being indicated on the pressure gage air content scale for the Type B meter. A change in elevation of more than 600 ft (183 m) from the location at which a Type-A meter was last calibrated will require recalibration in accordance with X1.7.

5. Determination of Aggregate Correction Factor

5.1 *Procedure*—Determine the aggregate correction factor on a combined sample of fine and coarse aggregate as directed in 5.2 to 5.4. It is determined independently by applying the calibrated pressure to a sample of inundated fine and coarse aggregate in approximately the same moisture condition, amount, and proportions occurring in the concrete sample under test.

5.2 *Aggregate Sample Size*—Calculate the weights of fine and coarse aggregate present in the sample of fresh concrete whose air content is to be determined, as follows:

$$F_s = (S/B) \times F_b \qquad (1)$$

$$C_s = (S/B) \times C_b \qquad (2)$$

where:

F_s = weight of fine aggregate in concrete sample under test, lb (kg),

S = volume of concrete sample (same as volume of measuring bowl), ft^3 (m^3),

B = volume of concrete produced per batch (Note 1), ft^3 (m^3),

F_b = total weight of fine aggregate in the moisture condition used in batch, lb (kg),

C_s = weight of coarse aggregate in concrete sample under test, lb (kg), and

C_b = total weight of coarse aggregate in the moisture condition used in batch, lb (kg).

NOTE 1—The volume of concrete produced per batch can be determined in accordance with applicable provisions of Test Method C 138.

NOTE 2—The term "weight" is temporarily used in this standard because of established trade usage. The word is used to mean both "force" and "mass," and care must be taken to determine which is meant in each case (SI unit for force = newton and for mass = kilogram).

5.3 *Placement of Aggregate in Measuring Bowl*—Mix representative samples of fine aggregate F_s, and coarse aggregate C_s, and place in the measuring bowl filled one-third full with water. Place the mixed aggregate, a small amount at a time, into the measuring bowl; if necessary, add additional water so as to inundate all of the aggregate. Add each scoopful in a manner that will entrap as little air as possible and remove accumulations of foam promptly. Tap the sides of the bowl and lightly rod the upper 1 in. (25 mm) of the aggregate about ten times. Stir after each addition of aggregate to eliminate entrapped air.

5.4 *Aggregate Correction Factor Determination*:

5.4.1 *Initial Procedure for Types A and B Meters*—When all of the aggregate has been placed in the measuring bowl, remove excess foam and keep the aggregate inundated for a period of time approximately equal to the time between introduction of the water into the mixer and the time of performing the test for air content before proceeding with the determination as directed in 5.4.2 or 5.4.3.

5.4.2 *Type A Meter*—Complete the test as described in 7.2.1 and 7.2.2. The aggregate correction factor, G, is equal to $h_1 - h_2$ (see Fig. 1) (Note 3).

5.4.3 *Type B Meter*—Perform the procedures as described in 7.3.1. Remove a volume of water from the assembled and filled apparatus approximately equivalent to the volume of air that would be contained in a typical concrete sample of a size equal to the volume of the bowl. Remove the water in the manner described in X1.9 of the appendix for the calibration tests. Complete the test as described in 7.3.2. The aggregate correction factor, G, is equal to the reading on the air-content scale minus the volume of water removed from the bowl expressed as a percent of the volume of the bowl (see Fig. 1).

NOTE 3—The aggregate correction factor will vary with different aggregates. It can be determined only by test, since apparently it is not directly related to absorption of the particles. The test can be easily made and must not be ignored. Ordinarily the factor will remain reasonably constant for given aggregates, but an occasional check test is recommended.

6. Preparation of Concrete Test Sample

6.1 Obtain the sample of freshly mixed concrete in accordance with applicable procedures of Method C 172. If the concrete contains coarse aggregate particles that would be retained on a 2-in. (50-mm) sieve, wet-sieve a sufficient amount of the representative sample over a 1½-in. (37.5-mm) sieve, as described in Method C 172, to yield somewhat more than enough material to fill the measuring bowl of the size selected for use. Carry out the wet-sieving operation with the minimum practicable disturbance of the mortar. Make no attempt to wipe adhering mortar from coarse aggregate particles retained on the sieve.

7. Procedure for Determining Air Content of Concrete

7.1 *Placement and Consolidation of Sample*:

7.1.1 Place a representative sample of the concrete, prepared as described in Section 6, in the measuring bowl in equal layers. Consolidate each layer by the rodding procedure (7.1.2) or by vibration (7.1.3). Strike-off the finally consolidated layer (7.1.4). Vibration shall not be employed to consolidate concrete having a slump greater than 3 in. (76 mm).

7.1.2 *Rodding*—Place the concrete in the measuring bowl in three layers of approximately equal volume. Consolidate each layer of concrete by 25 strokes of the tamping rod evenly distributed over the cross section. After each layer is rodded, tap the sides of the measure smartly 10 to 15 times with the mallet to close any voids left by the tamping rod and to release any large bubbles of air that may have been trapped. Rod the bottom layer throughout its depth, but the rod shall not forcibly strike the bottom of the measure. In rodding the second and final layers, use only enough force to cause the rod to penetrate the surface of the previous layer about 1 in. (25 mm). Add the final layer of concrete in a manner to avoid excessive overfilling (7.1.4).

7.1.3 *Vibration*—Place the concrete in the measuring bowl in two layers of approximately equal volume. Place all of the concrete for each layer before starting vibration of that layer. Consolidate each layer by three insertions of the vibrator evenly distributed over the cross section. Add the final layer in a manner to avoid excessive overfilling (7.1.4). In consolidating the bottom layer, do not allow the vibrator to rest on or touch the bottom or sides of the measuring bowl. Take care in withdrawing the vibrator to ensure that no air pockets are left in the specimen. Observe a standard duration of vibration for the particular kind of concrete, vibrator, and measuring bowl involved. The duration of vibration required will depend upon the workability of the concrete and the effectiveness of the vibrator. Continue vibration only long enough to achieve proper consolidation of the concrete. Overvibration may cause segregation and loss of intentionally entrained air. Usually, sufficient vibration has been applied as soon as the surface of the concrete becomes relatively smooth and has a glazed appearance. Never continue vibration long enough to cause escape of froth from the sample.

7.1.4 *Strike Off*—After consolidation of the concrete, strike off the top surface by sliding the strike-off bar across the top flange or rim of the measuring bowl with a sawing motion until the bowl is just level full. On completion of consolidation, the bowl must not contain a great excess or deficiency of concrete. Removal of approximately ⅛ in. (3 mm) during strike off is optimum. A small quantity of representative concrete may be added to correct a deficiency. If the measure contains a great excess, remove a representative portion of concrete with a trowel or scoop before the measure is struck off.

7.1.5 *Application of Test Method*—Any portion of the test method not specifically designated as pertaining to Type A or Type B meter shall apply to both types.

7.2 *Procedure—Type A Meter*:

7.2.1 *Preparation for Test*—Thoroughly clean the flanges or rims of the bowl and of the cover assembly so that when the cover is clamped in place a pressure-tight seal will be obtained. Assemble the apparatus and add water over the concrete by means of the tube until it rises to about the halfway mark in the standpipe. Incline the apparatus assembly about 30° from vertical and, using the bottom of the bowl as a pivot, describe several complete circles with the upper end of the column, simultaneously tapping the cover lightly to remove any entrapped air bubbles above the concrete sample. Return the apparatus assembly to a vertical position and fill the water column slightly above the zero mark, while lightly tapping the sides of the bowl. Bring the water level to the zero mark of the graduated tube before closing the vent at the top of the water column (see Fig. 1 A).

NOTE 4—The internal surface of the cover assembly should be kept clean and free from oil or grease; the surface should be wet to prevent adherence of air bubbles that might be difficult to dislodge after assembly of the apparatus.

7.2.2 *Test Procedure*—Apply slightly more than the desired test pressure, P, (about 0.2 psi (1380 Pa) more) to the concrete by means of the small hand pump. To relieve local restraints, tap the sides of the measure sharply and, when the pressure gage indicates the exact test pressure, P, as determined in accordance with X1.7, read the water level, h_1, and record to the nearest division or half-division on the graduated precision-bore tube or gage glass of the standpipe (see Fig. 1 B). For extremely harsh mixes it may be necessary to tap the bowl vigorously until further tapping produces no change in the indicated air content. Gradually release the air pressure through the vent at the top of the water column and tap the sides of the bowl lightly for about 1 min. Record the water level, h_2, to the nearest division or half-division (see Fig. 1 C). Calculate the apparent air content as follows:

$$A_1 = h_1 - h_2$$

where:

A_1 = apparent air content,
h_1 = water level reading at pressure, P (see Note 5), and
h_2 = water level reading at zero pressure after release of pressure, P.

7.2.3 *Check Test*—Repeat the steps described in 7.2.2 without adding water to reestablish the water level at the zero mark. The two consecutive determinations of apparent air content should check within 0.2 % of air and shall be averaged to give the value A to be used in calculating the air content, A_s, in accordance with Section 8.

7.2.4 In the event the air content exceeds the range of the meter when it is operated at the normal test pressure P, reduce the test pressure to the alternative test pressure P_1 and repeat the steps outlined in 7.2.2 and 7.2.3.

NOTE 5—See X1.7 for exact calibration procedures. An approximate value of the alternative pressure, P_1, such that the apparent air content will equal twice the meter reading can be computed from the following relationship:

$$P_1 = P_a P/(2P_a + P)$$

where:

P_1 = alternative test pressure, psi (or kPa),
P_a = atmospheric pressure, psi (approximately 14.7 psi (101 kPa) but will vary with altitude and weather conditions) (or kPa), and
P = normal test or operating gage pressure, psi (or kPa).

7.3 *Procedure—Type B Meter*.

7.3.1 *Preparation for Test*: Thoroughly clean the flanges or rims of the bowl and the cover assembly so that when the cover is clamped in place a pressure-tight seal will be obtained. Assemble the apparatus. Close the air valve between the air chamber and the measuring bowl and open both petcocks on the holes through the cover. Using a rubber syringe, inject water through one petcock until water emerges from the opposite petcock. Jar the meter gently until all air is expelled from this same petcock.

7.3.2 *Test Procedure*—Close the airbleeder valve on the air chamber and pump air into the air chamber until the gage hand is on the initial pressure line. Allow a few seconds for the compressed air to cool to normal temperature. Stabilize the gage hand at the initial pressure line by pumping or bleeding-off air as necessary, tapping the gage lightly. Close both petcocks on the holes through the cover. Open the air valve between the air chamber and the measuring bowl. Tap the sides of the measuring bowl sharply to relieve local restraints. Lightly tap the pressure gage to stabilize the gage hand and read the percentage of air on the dial of the pressure gage. Failure to close the main air valve before releasing the pressure from either the container or the air chamber will result in water being drawn into the air chamber, thus introducing error in subsequent measurements. In the event water enters the air chamber it must be bled from the air chamber through the bleeder valve followed by several strokes of the pump to blow out the last traces of water. Release the pressure by opening both petcocks (Fig. 1, A and B) before removing the cover.

8. Calculation

8.1 *Air Content of Sample Tested*—Calculate the air content of the concrete in the measuring bowl as follows:

$$A_s = A_1 - G \tag{3}$$

where:

A_s = air content of the sample tested, %,

A_1 = apparent air content of the sample tested, % (see 7.2.2 and 8.3.2), and

G = aggregate correction factor, % (Section 5).

8.2 *Air Content of Full Mixture*—When the sample tested represents that portion of the mixture that is obtained by wet sieving to remove aggregate particles larger than a 1½-in. (37.5-mm) sieve, the air content of the full mixture may be calculated as follows:

$$A_t = 100\, A_s V_c / (100\, V_t - A_s V_a) \tag{4}$$

where: (Note 6)

A_t = air content of the full mixture, %,

V_c = absolute volume of the ingredients of the mixture passing a 1½-in. sieve, airfree, as determined from the original batch weights, ft³ (m³),

V_t = absolute volume of all ingredients of the mixture, airfree, ft³ (m³), and

V_a = absolute volume of the aggregate in the mixture coarser than a 1½-in. sieve, as determined from original batch weights, ft³ (m³).

8.3 *Air Content of the Mortar Fraction*—When it is desired to know the air content of the mortar fraction of the mixture, calculate it as follows:

$$A_m = 100\, A_s V_c / [100\, V_m + A_s (V_c - V_m)] \tag{5}$$

where: (Note 6)

A_m = air content of the mortar fraction, %, and

V_m = absolute volume of the ingredients of the mortar fraction of the mixture, airfree, ft³ (m³).

NOTE 6—The values for use in Eqs 4 and 5 are most conveniently obtained from data on the concrete mixture tabulated as follows for a batch of any size:

	Absolute Volume, ft³ (m³)	
Cement	——	V_m, V_c
Water	——	
Fine aggregate	——	
Coarse aggregate (No. 4 (4.75-mm) to 1½-in. (37.5-mm))	——	
Coarse aggregate (1½-in.)	——	V_a
Total	——	V_t

9. Precision

9.1 Data are being compiled and developed that will be suitable for use in developing precision statements for this method.

APPENDIX

X1. CALIBRATION OF APPARATUS

X1.1 Calibration tests shall be performed in accordance with the following procedures as applicable to the meter type being employed.

X1.2 *Calibration of the Calibration Vessel*—Determine accurately the weight of water, *w*, required to fill the calibration vessel, using a scale accurate to 0.1 % of the weight of the vessel filled with water. This step shall be performed for Type A and B meters.

X1.3 *Calibration of the Measuring Bowl*—Determine the weight of water, *W*, required to fill the measuring bowl, using a scale accurate to 0.1 % of the weight of the bowl filled with water. Slide a glass plate carefully over the flange of the bowl in a manner to ensure that the bowl is completely filled with water. A thin film of cup grease smeared on the flange of the bowl will make a watertight joint between the glass plate and the top of the bowl. This step shall be performed for Type A and B meters.

X1.4 *Effective Volume of the Calibration Vessel, R*—The constant *R* represents the effective volume of the calibration vessel expressed as a percentage of the volume of the measuring bowl.

X1.4.1 For meter Types A, calculate *R* as follows (Note X1):

$$R = 0.98\, w/W \tag{X1}$$

where:

w = weight of water required to fill the calibration vessel, and

W = weight of water required to fill the measuring bowl.

NOTE X1—The factor 0.98 is used to correct for the reduction in the volume of air in the calibration vessel when it is compressed by a depth of water equal to the depth of the measuring bowl. This factor is approximately 0.98 for an 8-in. (203-mm) deep measuring bowl at sea level. Its value decreases to approximately 0.975 at 5000 ft (1524 m) above sea level and 0.970 at 13 000 ft (3962 m) above sea level. The value of this constant will decrease by about 0.01 for each 4-in. (102-mm) increase in bowl depth. The depth of the measuring bowl and atmospheric pressure do not affect the effective volume of the calibration vessel for meter Types B.

X1.4.2 For meter Types B calculate R as follows (Note X1):

$$R = w/W \tag{X2}$$

X1.5 *Determination of, or Check of, Allowance for Expansion Factor, D:*

X1.5.1 For meter assemblies of Type A determine the expansion factor, D (Note X2) by filling the apparatus with water only (making certain that all entrapped air has been removed and the water level is exactly on the zero mark (Note X3) and applying an air pressure approximately equal to the operating pressure, P, determined by the calibration test described in X1.7. The amount the water column lowers will be the equivalent expansion factor, D, for that particular apparatus and pressure (Note X5).

NOTE X2—Although the bowl, cover, and clamping mechanism of the apparatus must of necessity be sturdily constructed so that it will be pressure-tight, the application of internal pressure will result in a small increase in volume. This expansion will not affect the test results because, with the procedure described in Sections 5 and 7, the amount of expansion is the same for the test for air in concrete as for the test for aggregate correction factor on com[illegible]ed fine and coarse aggregates, and is thereby automatically cancell[illegible]. However, it does enter into the calibration test to determine the air pressure to be used in testing fresh concrete.

NOTE X3—The water columns on some meters of Type-A design are marked with an initial water level and a zero mark, the difference between the two marks being the allowance for the expansion factor. This allowance should be checked in the same manner as for meters not so marked and in such a case, the expansion factor should be omitted in computing the calibration readings in X1.7.

NOTE X4—It will be sufficiently accurate for this purpose to use an approximate value for P determined by making a preliminary calibration test as described in X1.7 except that an approximate value for the calibration factor, K, should be used. For this test $K = 0.98R$ which is the same as Eq X2 except that the expansion reading, D, as yet unknown, is assumed to be zero.

X1.5.2 For meters of Type B design, the allowance for the expansion factor, D, is included in the difference between the initial pressure indicated on the pressure gage and the zero percent mark on the air-content scale on the pressure gage. This allowance shall be checked by filling the apparatus with water (making certain that all entrapped air has been removed), pumping air into the air chamber until the gage hand is stabilized at the indicated initial pressure line, and then releasing the air to the measuring bowl (Note X5). If the initial pressure line is correctly positioned, the gage should read zero percent. The initial pressure line shall be adjusted if two or more determinations show the same variation from zero percent and the test repeated to check the adjusted initial pressure line.

NOTE X5—This procedure may be accomplished in conjunction with the calibration test described in X1.9.

X1.6 *Calibration Reading, K*—The calibration reading, K, is the final meter reading to be obtained when the meter is operated at the correct calibration pressure.

X1.6.1 For meter Types A, the calibration reading, K, is as follows:

$$K = R + D \tag{X3}$$

where:

R = effective volume of the calibration vessel (X4.1), and
D = expansion factor (X5.1, Note X6).

X1.6.2 For meter Types B the calibration reading, K, equals the effective volume of the calibration vessel (X4.2) as follows:

$$K = R \tag{X4}$$

NOTE X6—If the water column indicator is graduated to include an initial water level and a zero mark, the difference between the two marks being equivalent to the expansion factor, the term D shall be omitted from Eq X3.

X1.7 *Calibration Test to Determine Operating Pressure, P, on Pressure Gage, Type A Meter*—If the rim of the calibration cylinder contains no recesses or projections, fit it with three or more spacers equally spaced around the circumference. Invert the cylinder and place it at the center of the dry bottom of the measuring bowl. The spacers will provide an opening for flow of water into the calibration cylinder when pressure is applied. Secure the inverted cylinder against displacement and carefully lower the cover assembly. After the cover is clamped in place, carefully adjust the apparatus assembly to a vertical position and add water at air temperature, by means of the tube and funnel, until it rises above the zero mark on the st[illegible]ndpipe. Close the vent and pump air into the apparatus t[illegible] the approximate operating pressure. Incline the assembly about 30° from vertical and, using the bottom of the bowl as a pivot, describe several complete circles with the upper end of the standpipe, simultaneously tapping the cover and sides of the bowl lightly to remove any entrapped air adhering to the inner surfaces of the apparatus. Return the apparatus to a vertical position, gradually release the pressure (to avoid loss of air from the calibration vessel), and open the vent. Bring the water level exactly to the zero mark by bleeding water through the petcock in the top of the conical cover. After closing the vent, apply pressure until the water level has dropped an amount equivalent to about 0.1 to 0.2 % of air more than the value of the calibration reading, K, determined as described in X1.6. To relieve local restraints, lightly tap the sides of the bowl, and when the water level is exactly at the value of the calibration reading, K, read the pressure, P, indicated by the gage and record to the nearest 0.1 psi (690 Pa). Gradually release the pressure and open the vent to determine whether the water level returns to the zero mark when the sides of the bowl are tapped lightly (failure to do so indicates loss of air from the calibration vessel or loss of water due to a leak in the assembly). If the water levels fails to return to within 0.05 % air of the zero mark and no leakage beyond a few drops of water is found, some air probably was lost from the calibration cylinder. In this case, repeat the calibration procedure step by step from the beginning of this paragraph. If the leakage is more than a few drops of water, tighten the leaking joint before repeating the calibration procedure. Check the indicated pressure reading promptly by bringing the water level exactly to the zero mark, closing the vent, and applying the pressure, P, just

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determined. Tap the gage lightly with a finger. When the gage indicates the exact pressure, P, the water column should read the value of the calibration factor, K, used in the first pressure application within about 0.05 % of air. **Caution**—The apparatus assembly must not be moved from the vertical position until pressure has been applied which will force water about one third of the way up into the calibration cylinder. Any loss of air from this cylinder will nullify the calibration.

X1.8 *Calibration Test to Determine Alternative Operating Pressure P_l—Meter Type A*—The range of air contents which can be measured with a given meter can be doubled by determining an alternative operating pressure P_1 such that the meter reads half of the calibration reading, K, (Eq. X3). Exact calibration will require determination of the expansion factor at the reduced pressure in X1.5. For most purposes the change in expansion factor can be disregarded and the alternative operating pressure determined during the determination of the regular operating pressure in X1.7.

X1.9 *Calibration Test to Check the Air Content Graduations on the Pressure Gage, Type B Meter*—Fill the measuring bowl with water as described in X1.3. Screw the short piece of tubing or pipe furnished with the apparatus into the threaded petcock hole on the underside of the cover assembly. Assemble the apparatus. Close the air valve between the air chamber and the measuring bowl and open the two petcocks on holes through the cover assembly. Add water through the petcock on the cover assembly having the extension below until all air is expelled from the second petcock. Pump air into the air chamber until the pressure reaches the indicated initial pressure line. Allow a few seconds for the compressed air to cool to normal temperature. Stabilize the gage hand at the initial pressure line by pumping or bleeding off air as necessary, tapping the gage lightly. Close the petcock not provided with the tube or pipe extension on the under side of the cover. Remove water from the assembly to the calibrating vessel controlling the flow, depending on the particular meter design, by opening the petcock provided with the tube or pipe extension and cracking the air valve between the air chamber and the measuring bowl, or by opening the air valve and using the petcock to control flow. Perform the calibration at an air content which is within the normal range of use. If the calibration vessel (X1.2) has a capacity within the normal range of use, remove exactly that amount of water. With some meters the calibrating vessel is quite small and it will be necessary to remove several times that volume to obtain an air content within the normal range of use. In this instance, carefully collect the water in an auxiliary container and determine the amount removed by weighing to the nearest 0.1 %. Calculate the correct air content, R, by using Eq X2. Release the air from the apparatus at the petcock not used for filling the calibration vessel and if the apparatus employs an auxiliary tube for filling the calibration container, open the petcock to which the tube is connected to drain the tube back into the measuring bowl (Note X8). At this point of procedure the measuring bowl contains the percentage of air determined by the calibration test of the calibrating vessel. Pump air into the air chamber until the pressure reaches the initial pressure line marked on the pressure gage, close both petcocks in the cover assembly, and then open the valve between the air chamber and the measuring bowl. The indicated air content on the pressure gage dial should correspond to the percentage of air determined to be in the measuring bowl. If two or more determinations show the same variation from the correct air content, the dial hand shall be reset to the correct air content and the test repeated until the gage reading corresponds to the calibrated air content within 0.1 %. If the dial hand was reset to obtain the correct air content, recheck the initial pressure mark as in X1.5.2. If a new initial pressure reading is required, repeat the calibration to check the accuracy of the graduation on the pressure gage described earlier in this section. If difficulty is encountered in obtaining consistent readings, check for leaks, for the presence of water inside the air chamber (see Fig. 2), or the presence of air bubbles clinging to the inside surfaces of the meter from the use of cool aerated water. In this latter instance use deaerated water which can be obtained by cooling hot water to room temperature.

Note X7—If the calibrating vessel is an integral part of the cover assembly, the petcock used in filling the vessel should be closed immediately after filling the calibration vessel and not opened until the test is complete.

Standard Test Method for Temperature of Freshly Mixed Portland-Cement Concrete[1]

This standard is issued under the fixed designation C 1064; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This test method covers the determination of temperature of freshly mixed portland cement concrete.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 172 Method for Sampling Freshly Mixed Concrete[2]

E 1 Specification for ASTM Thermometers[3]

E 77 Method for Verification and Calibration of Liquid-in-Glass Thermometers[3]

2.2 *NBS Standard:*

N.B.S. Monograph 150 Liquid-in-Glass Thermometry[4]

3. Significance and Use

3.1 This test method provides a means for measuring the temperature of freshly mixed concrete. It may be used to verify conformance to a specified requirement for temperature of concrete.

3.2 Concrete containing aggregate of a nominal maximum size greater than 3 in. (75 mm) may require up to 20 min for the transfer of heat from aggregate to mortar. (See ACI Committee 207.1R Report.[5])

4. Apparatus

4.1 *Container*—The container shall be made of nonabsorptive material and large enough to provide at least 3 in. (75 mm) of concrete in all directions around the sensor of the temperature measuring device; concrete cover must also be at least three times the nominal maximum size of the coarse aggregate.

4.2 *Temperature Measuring Device*—The temperature measuring device shall be capable of measuring the temperature of the freshly mixed concrete to ±1°F (±0.5°C) throughout the entire temperature range likely to be encountered in the fresh concrete. ASTM liquid-in-glass thermometers having a range from 0 to 120°F (−18 to 49°C), and conforming to the requirements for ASTM thermometer No. 36°C as prescribed in Specification E 1 are satisfactory. Other thermometers of the required accuracy, including the metal immersion type, are acceptable.

4.3 Partial immersion liquid-in-glass thermometers (and possibly other types) shall have a permanent mark to which the device must be immersed without applying a correction factor.

4.4 *Reference Temperature Measuring Device*—The reference temperature measuring device shall be a liquid-in-glass thermometer readable to 0.5°F (0.2°C) that has been verified and calibrated in accordance with Method E 77. The calibration certificate or report shall be available for inspection.

5. Calibration of Temperature Measuring Device

5.1 Each temperature measuring device used for determining temperature of freshly mixed concrete shall be calibrated annually, or whenever there is a question of accuracy. This calibration shall be performed by comparing the readings on the temperature measuring device at two temperatures at least 30°F (15°C) apart.

5.2 Calibration of the temperature measuring devices may be made in oil or other suitable baths having uniform density if provision is made to:

5.2.1 Maintain the bath temperature constant within 0.5°F (0.2°C) during the period of the test.

5.2.2 Have both the temperature and reference temperature measuring devices maintained in the bath for a minimum of 5 min before reading temperatures.

5.2.3 Continuously circulate the bath liquid to provide a uniform temperature.

5.2.4 Slightly tap thermometers containing liquid to avoid adhesion of the liquid to the glass if the temperature exposure is being reduced.

5.3 If a limiting temperature is specified, calibrate the measuring device at a temperature within ±5°F (2°C) of the limiting temperature permitted.

6. Sampling Concrete

6.1 The temperature of freshly mixed concrete may be measured in the transporting equipment provided the sensor of the temperature measuring device has at least 3 in. (75 mm) of concrete cover in all directions around it.

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[1] This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and are the direct responsibility of Subcommittee C09.03.03 on Methods of Testing Fresh Concrete.

Current edition approved July 17, 1986. Published July 1986.

[2] *Annual Book of ASTM Standards*, Vol 04.02.

[3] *Annual Book of ASTM Standards*, Vol 14.01.

[4] Available from National Bureau of Standards, U.S. Department of Commerce, Washington, DC 20234.

[5] Available from American Concrete Institute, Box 19150, Redford Station, Detroit, MI 48219. Other related documents also available from American Concrete Institute are Committee Reports 305, (Hot Weather Concreting) and 306 (Cold Weather Concreting).

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6.2 Temperature of the freshly mixed concrete may be obtained following concrete placement using the forms as the container.

6.3 If the transporting equipment or placement forms are not used as the container, a sample shall be prepared as follows:

6.3.1 Immediately, prior to sampling the freshly mixed concrete, dampen (with water) the sample container.

6.3.2 Sample the freshly mixed concrete in accordance with Method C 172, except that composite samples are not required if the only purpose for obtaining the sample is to determine temperature.

6.3.3 Place the freshly mixed concrete into the container.

6.3.4 When concrete contains a nominal maximum size of aggregate greater than 3 in. (75 mm), it may require 20 min before the temperature is stabilized after mixing.

7. Procedure

7.1 Place the temperature measuring device in the freshly mixed concrete so that the temperature sensing portion is submerged a minimum of 3 in. (75 mm). Gently press the concrete around the temperature measuring device at the surface of the concrete so that ambient air temperature does not affect the reading.

7.2 Leave the temperature measuring device in the freshly mixed concrete for a minimum period of 2 min or until the temperature reading stabilizes, then read and record the temperature.

7.3 Complete the temperature measurement of the freshly mixed concrete within 5 min after obtaining the sample.

8. Report

8.1 Record the measured temperature of the freshly mixed concrete to the nearest °F (0.5°C).

9. Precision and Bias

9.1 The precision and bias of this test method have not been determined. A precision and bias statement will be included when sufficient test data have been obtained and analyzed.

Designation: E 329 – 77 (Reapproved 1983)

Standard Recommended Practice for Inspection and Testing Agencies for Concrete, Steel, and Bituminous Materials as Used in Construction[1]

This standard is issued under the fixed designation E 329; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This recommended practice has been approved for use by agencies of the Department of Defense and for listing in the DoD Index of Specifications and Standards.

INTRODUCTION

Purpose

The purpose of inspection and testing of materials, composites, and practices is to determine whether or not their characteristics and qualities as used in construction comply with the contract documents.

This recommended practice is intended not only to describe the requirements and functions of the agency in achieving this purpose including complete reliability reports, but also to indicate relationships between the agency and the other parties concerned.

Relationships

The agency shall be selected and authorized by the owner or his representative to perform the prescribed tests and inspection.

The agency must be given written authority by the owner or his representative to perform the prescribed tests and inspections. The agency shall be authorized in writing by the owner or his representative to have free access to the site to the shops or yards where materials are being prepared or stored, as well as to any relevant data on previous testing and investigations of the materials. The agency shall be provided with all applicable plans, specifications, addenda, change orders, shop drawings, and other necessary information.

Full cooperation between the agency and the other parties concerned is necessary to ensure proper inspection and testing with minimum interference or delay in the work.

1. Scope

1.1 This recommended practice defines duties and responsibilities and establishes minimum requirements for personnel and equipment of public and independent commercial materials inspection and testing agencies engaged in inspection and testing of concrete, steel, and bituminous materials as used in construction.

1.2 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Description of Terms

2.1 *Concrete*—For the purpose of this recommended practice, concrete, as used in construction, embraces all portland cement concrete used in construction, particularly reinforced concrete.

2.2 *Steel*—For the purposes of this recommended practice, steel, as used in construction, embraces structural steel plates and shapes used wholly or in part for structures. It also includes reinforcing steel used in concrete. It is not intended to include steels used in conjunction with mechanical, electrical, heating or air-conditioning equipment except for the supporting steel structures.

2.3 *Bituminous Materials and Mixes*—For the purposes of this recommended practice, bituminous materials, as used in construction, include all types of asphalts and tars for pavements. Bituminous mixes are bituminous stabilized soil, base course, binder, leveling, surface course, and mastic mixes. Bituminous mixes may contain either tar or asphalt binder material which, in some cases, may be further modified with other additive materials to produce special properties.

2.4 *Agency*—The organization authorized by the owner or his duly authorized representative, to inspect or test concrete, steel, or bituminous materials as required by the specifications.

2.5 *Authority*—The owner, the engineer, or the architect in responsible charge of the work or his duly recognized or authorized representative.

[1] This recommended practice is under the jurisdiction of ASTM Committee E-36 on Criteria for the Evaluation of Testing and Inspection Agencies.

This recommended practice was circulated for review before acceptance to the American Concrete Inst.; American Society of Civil Engineers; American Council of Independent Laboratories; Construction Specifications Inst.; and a joint AIA-ASCE-ACI Committee on Quality in Concrete.

Current edition approved Nov. 10, 1977. Published February 1978. Originally published as E 329 – 67 T. Last previous edition E 329 – 72.

2.6 *Technician*—An employee of the inspection and testing agency assigned to perform the actual operations of inspection or testing.

2.7 *Nondestructive Testing*—For the purpose of this recommended practice nondestructive testing includes all test methods that do not impair the serviceability of the material, part, or assembly under test. Nondestructive tests are specific. They usually reveal only the specific kinds of defects and conditions for whose detection they were designed. Consequently they must be selected in accordance with the specific materials, the specific conditions to be detected, and the specific job to be done.

3. Responsibilities and Duties

3.1 It shall be the responsibility of the agency to ensure that it performs only inspections or tests for which it is adequately equipped and staffed and that its employees perform only inspections and tests for which they are adequately trained.

3.2 The following duties are those usually performed by the agency:

3.2.1 Obtain representative samples of those materials required by project specifications to be tested and evaluated.

3.2.2 Ascertain that there is proper protection, curing, handling, and storing of the samples to assure that they remain representative of the material being used at the time of sampling.

3.2.3 Ascertain that the samples are identified with the respective portions of the work in which the material represented was or will be used.

3.2.4 Perform all testing and inspection operations in accordance with appropriate standards.

3.2.5 Call to the attention of the proper authority at once any irregularity or deficiency.

3.2.6 Submit promptly to the proper authority formal reports of all tests and inspections which indicate compliance or noncompliance with the specifications. The reports shall be complete and factual, citing the methods used in obtaining samples, the tests performed, the specified values for the measured characteristics, the values obtained, the parts of the structure involved, and similar pertinent data. The agency, realizing the seriousness of its reports, shall be prepared to substantiate them to the fullest extent.

3.2.7 Ascertain that the construction plant and equipment meet the specifications and are operated in conformity therewith.

3.3 Unless specifically authorized, the agency does not have the right of rejection.

3.4 The agency shall have its laboratory procedures and equipment inspected at intervals of not more than 3 years by a qualified national authority as evidence of its competence to perform the required tests.

NOTE 1—The Materials Reference Laboratories at the National Bureau of Standards are such qualified national authorities.

3.5 The qualified national authority, when making an inspection, shall obtain an affidavit as to the qualification of the personnel as listed in Section 4.

4. Management and Supervision

4.1 The inspection and testing services of the agency shall be under the direction of a person charged with engineering managerial responsibility. He shall be a registered engineer and a full-time employee of that agency. He shall have at least 5 years' engineering experience in inspection and testing of construction and materials.

4.2 A supervising laboratory technician shall have at least 5 years' experience performing tests on construction materials. He shall be able to demonstrate his ability to perform the tests normally required in the manner stipulated under ASTM or other governing procedures.

4.3 A supervising field technician shall have at least 5 years' inspection experience in the kind of work involved on construction projects. He shall be able to demonstrate either by oral or written examination, or both, his ability to perform correctly the duties required of him.

CONCRETE INSPECTION AND TESTING

5. General

5.1 Concrete inspection and testing services will normally include some or all of the following: sampling and testing of ingredients; mix design; checking of production equipment and procedures; inspection of placement and curing; and laboratory testing of hardened specimens. The provisions of Sections 1 to 4 supplement these sections.

6. Equipment for Concrete Inspection and Testing

6.1 *Laboratory Equipment*—The laboratory of the agency responsible for testing concrete shall be equipped with at least the following:

6.1.1 A screw- or hydraulic-type of compression testing machine of sufficient capacity to test any specimen which may be involved in the construction (normally a machine with at least 200 000-lb (91 000-kg) capacity). It shall conform to all requirements of ASTM Practices E 4, Load Verification of Testing Machines,[2] and ASTM Test Method C 39, for Compressive Strength of Cylindrical Concrete Specimens.[3] The machine shall be verified annually in accordance with Practices E 4 and a report giving details of the verification shall be readily available.

6.1.2 Adequate facilities for preparing concrete test specimens in accordance with ASTM Method C 192, Making and Curing Concrete Test Specimens in the Laboratory.[3]

6.1.3 Reusable cylinder molds conforming to Method C 192 or single-use molds conforming to ASTM Specification C 470, Molds for Forming Concrete Test Cylinders Vertically.[3]

6.1.4 Adequate facilities for curing concrete specimens in accordance with Method C 192. These facilities may consist of either a thermostatically controlled fog room with required temperature and humidity control or thermostatically

[2] *Annual Book of ASTM Standards*, Vols 03.01, 04.02, 07.01, and 08.03.
[3] *Annual Book of ASTM Standards*, Vol 04.02.

controlled tanks containing saturated lime solution.

6.1.5 Adequate facilities for capping test specimens in accordance with Method C 617, Capping Cylindrical Concrete Specimens.[3]

6.1.6 Adequate facilities for performing the test for flexural strength of concrete in accordance with Test Method C 78, for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading).[3]

6.1.7 Equipment for testing ingredient materials conforming to the following ASTM methods:

- C 131, Test Method for Resistance Degradation of Small Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine,[3]
- C 142, Test Method for Clay Lumps and Friable Particles in Aggregates,[3]
- C 123, Test Method for Lightweight Pieces in Aggregate,[3]
- C 117, Test Method for Materials Finer Than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing,[3]
- C 40, Test Method for Organic Impurities in Fine Aggregates for Concrete,[3]
- C 136, Method for Sieve Analysis of Fine and Coarse Aggregates,[3]
- C 88, Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate,[3]
- C 127, Test Method for Specific Gravity and Absorption of Coarse Aggregate,[3]
- C 128, Test Method for Specific Gravity and Absorption of Fine Aggregate,[3]
- C 566, Test Method for Total Moisture Content of Aggregate by Drying,[3] and
- C 29, Test Method for Unit Weight and Voids in Aggregate.[3]

6.1.8 Access to facilities for physical and chemical analysis of cement.

6.1.9 Access to facilities for testing of curing compounds, admixtures, and related materials.

6.2 *Field Inspection Equipment*—A pocket thermometer and equipment conforming to requirements of the following ASTM methods:

- C 231, Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method,[3]
- C 173, Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method,[3]
- C 360, Test Method for Ball Penetration in Fresh Portland Cement Concrete,[7]
- C 31, Making and Curing Concrete Test Specimens in the Field,[3]
- C 172, Method of Sampling Freshly Mixed Concrete,[3]
- C 143, Test Method for Slump of Portland Cement Concrete,[3] and
- C 138, Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete.[3]

STEEL INSPECTION AND TESTING

7. General

7.1 Steel inspection and testing services will normally include one or more of the following general functions: inspection at source of base material (the steel mill); inspection at fabrication shop; inspection at erection site; laboratory testing to determine physical and chemical properties of steel; laboratory tests of paints for use on steel structures; qualification of welding procedures and personnel; nondestructive testing (radiographic, magnetic particle, dye penetrant, ultrasonic, etc.); and inspection of cutting and bending of reinforcing bars and testing of same. The provisions of Sections 1 to 4 supplement these sections.

8. Equipment for Steel Inspection and Testing

8.1 *Laboratory Equipment*—The laboratory of the agency responsible for testing steel shall be equipped with at least the following:

8.1.1 Suitable facilities for preparing test specimens.

8.1.2 A screw- or hydraulic-type of testing machine of sufficient capacity to test any specimen which may be involved (normally a multiple-range machine with at least 200 000-lb (91 000-kg) capacity). The machine shall be equipped with suitable gripping and bending tools and with variable speed control. It shall be verified annually in accordance with a procedure specified by Practices E 4 and shall meet the accuracy requirement of these methods. A report giving detail of the verification shall be readily available.

8.1.3 Hardness measuring device (Rockwell or Brinell).

8.1.4 Appropriate measuring equipment such as micrometers, rules, dividers, etc.

8.1.5 Access to a chemical laboratory suitably equipped for the analysis of constituents and alloying elements of structural steels, and for analysis of paints to applicable specification.

8.1.6 AWS standard guided-bend test jig.

8.1.7 Bend test jig for reinforcing steel.

8.2 *Field Equipment (Mill, Fabrication, and Erection)*—At least the following items of equipment appropriate to the service to be rendered shall be readily available:

8.2.1 Steel tape, rule, calipers, and other appropriate measuring equipment.

8.2.2 Weld dimension gage.

8.2.3 Weld viewing shield.

8.2.4 Hammer for weld testing.

8.2.5 Hammer and ball for rivet testing.

8.2.6 Strong hand light.

8.2.7 Paint thickness gage.

8.2.8 Thermometer (or temperature-measuring crayons).[4]

8.2.9 Inspector's identification stamp or tags.

8.2.10 Calibrated torque wrench for high strength bolts.

8.2.11 Device for calibrating impact and torque wrench.[5]

8.3 When nondestructive testing is required, the agency performing the testing shall meet the requirements of Practice E 543, for Determining the Qualification of Nondestruc-

[4] Tempilstiks available from Tempil Corp., 132 W 22 St., New York, NY, have been found satisfactory for this purpose.

[5] Skidmore-Wilhelm or equivalent.

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tive Testing Agencies,[6] in their entirety. The requirements of Practice E 543 supersede Sections 3 and 4 of this recommended practice with respect to nondestructive testing.

8.4 Unless otherwise specified, the tests performed shall be in accordance with the test methods specified in Section 2 of Practice E 543.

[6] *Annual Book of ASTM Standards*, Vol 03.03.

BITUMINOUS INSPECTION AND TESTING

9. General

9.1 Bituminous inspection and testing will normally include some or all of the following services:

9.1.1 Sampling of the bituminous material at the refinery, terminal, mix plant, or project site,

9.1.2 Testing of bituminous materials and mixtures in the laboratory,

9.1.3 Investigation of aggregate at source for compliance with the specification requirements,

9.1.4 Preparation of mix design with or without stability determinations such as Marshall, Hubbard-Field, Hveem, or others,

9.1.5 Inspection of proportioning and mixing at the plant or project site,

9.1.6 Determination of percent bitumen and grading of aggregates in plant mix,

9.1.7 Determination of stability in plant mix,

9.1.8 Inspection of spreading and rolling,

9.1.9 Determination of thickness of compacted mix, and

9.1.10 Determination of density of samples from compacted surface.

9.2 The provisions of Sections 1 to 4 supplement these sections.

10. Equipment for Bituminous Inspection and Testing

10.1 *Test Methods Applicable to Asphaltic Materials:*

10.1.1 *Penetrometer and Auxiliary Equipment:*

D 5, Test Method for Penetration of Bituminous Materials.[7]

10.1.2 *Viscometer and Auxiliary Equipment:*

D 88, Test Method for Saybolt Viscosity.[8]

D 2170, Test Method for Kinematic Viscosity of Asphalts (Bitumens).[7]

D 2171, Test Method for Viscosity of Asphalts by Vacuum Capillary Viscometer.[7]

E 102, Test Method for Saybolt Furol Viscosity of Bituminous Materials at High Temperatures.[10]

10.1.3 *Float Test Equipment:*

D 139, Method of Float Test for Bituminous Materials.[9]

10.1.4 *Softening Point Equipment:*

D 36, Test Method for Softening Point of Bitumen (Ring-and-Ball Apparatus).[10]

D 2398, Test Method for Softening Point of Bitumen in Ethylene Glycol (Ring-and-Ball).[10]

10.1.5 *Ductility Machine and Molds:*

D 113, Test Method for Ductility of Bituminous Materials.[7]

[7] *Annual Book of ASTM Standards*, Vol 04.03.

[8] *Annual Book of ASTM Standards*, Vols 04.04 and 10.03.

[9] *Annual Book of ASTM Standards*, Vols 04.03 and 04.08.

[10] *Annual Book of ASTM Standards*, Vol 04.04

10.1.6 *Flash Point Equipment:*

D 92, Test Method for Flash and Fire Points by Cleveland Open Cup.[11]

D 3143, Test Method for Flash Point of Cutback Asphalt with Tag Open-Cup Apparatus.[7]

10.1.7 *Effect of Heat and Air on Asphaltic Materials (Thin Film Test):*

D 1754, Test Method for Effect of Heat and Air on Asphaltic Materials (Thin-Film Oven Test).[7]

D 2872, Test Method for Effect of Heat and Air on a Moving Film of Asphalt (Rolling Thin-Film Oven Test).[7]

10.1.8 *Distillation Equipment:*

D 95, Test Method for Water in Petroleum Products and Bituminous Materials by Distillation.[12]

D 402, Test Method for Distillation of Cut-Back Asphaltic (Bituminous) Products.[7]

10.1.9 *Solubility Equipment:*

D 2042, Test Method for Solubility of Asphalt Materials in Trichloroethylene.[7]

10.1.10 *Specific Gravity Equipment:*

D 70, Test Method for Specific Gravity of Semi-Solid Bituminous Materials.[9]

D 3142, Test Method for Specific Gravity or API Gravity of Liquid Asphalts by Hydrometer Method.[7]

10.1.11 *Equipment for Residue of Specified Penetration:*

D 243, Test Method for Residue of Specified Penetration.[7]

10.1.12 *Equipment for Spot Test:*

AASHTO T102, Spot Test of Asphaltic Materials,

10.1.13 *Equipment for Testing Asphalt Emulsions:* The following tests of

D 244, Methods of Testing Emulsified Asphalts:[7] Residue by Distillation, Particle Charge, Saybolt Viscosity, Demulsibility, Settlement, Cement Mixing, Sieve Test, Coating Ability and Water Residue, Examination of Residue by Penetration, Ductility, and Solubility.

10.2 *Test Methods Applicable to Tar Products:*

10.2.1 *Viscometer and Auxiliary Equipment:*

D 1665, Test Method for Engler Specific Viscosity of Tar Products.[7]

10.2.2 *Float Test Equipment:*

D 139, Method of Float Test for Bituminous Materials.[9]

10.2.3 *Softening Point Equipment:*

D 36, Test Method for Softening Point of Bitumen (Ring-and-Ball Apparatus).[10]

D 2398, Test Method for Softening Point of Bitumen in Ethylene Glycol (Ring-and-Ball).[10]

10.2.4 *Distillation Equipment:*

D 20, Test Method for Distillation of Road Tars.[7]

[11] *Annual Book of ASTM Standards*, Vols 05.01 and 10.03.

[12] *Annual Book of ASTM Standards*, Vols 05.01, 06.01, and 06.03.

D 95, Test Method for Water in Petroleum Products and Bituminous Materials by Distillation.[12]

10.2.5 *Sulfonation Index Equipment:*

D 872, Test Method for Sulfonation Index of Road Tars.[7]

10.2.6 *Ductility Machine and Molds:*

D 113, Test Method for Ductility of Bituminous Materials.[7]

10.2.7 *Solubility Equipment:*

D 4, Test Method for Bitumen Content.[10]

D 2042, Test Method for Solubility of Asphalt Materials in Trichloroethylene.[10]

10.2.8 *Specific Gravity Equipment:*

D 70, Test Method for Specific Gravity of Semi-Solid Bituminous Materials.[9]

10.3 *Bituminous Mixes:*

10.3.1 *Extraction and Mix Gradation Equipment:*

D 2172, Test Methods for Quantitative Extraction of Bitumen from Bituminous Paving Mixtures.[7]

AASHTO T30, Mechanical Analysis of Extracted Aggregates.

10.3.2 *Stability and Compressive Strength Equipment (Note 2):*

D 1074, Test Method for Compressive Strength of Bituminous Mixtures.[9]

D 1075, Test Method for Effect of Water and Cohesion of Compacted Bituminous Mixtures.[7]

D 1138, Test Method for Resistance to Plastic Flow of Fine-Aggregate Bituminous Mixtures by Means of the Hubbard-Field Apparatus.[13]

D 1559, Test Method for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus.[7]

D 1560, Test Method for Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus.[7]

10.3.3 *Equipment for Specific Gravity of Mixes:*

D 1188, Test Method for Bulk Specific Gravity of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens.[9]

D 2041, Test Method for Theoretical Maximum Specific Gravity of Bituminous Paving Mixtures.[7]

D 2726, Test Method for Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens.[7]

10.3.4 *Equipment for Aggregate Gradation Tests:*

C 117, Test Method for Materials Finer Than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing.[7]

C 136, Test Method for Sieve or Screen Analysis of Fine and Coarse Aggregates.[7]

10.3.5 Other aggregate testing equipment as listed in Section 6 of this recommended practice or access to outside facilities furnishing such testing facilities is also required.

NOTE 2—The standards in 10.3.2 are alternative methods for evaluation of mix design, some common in some geographical areas, some in others. It is unlikely that any one laboratory would need to be equipped for all of these procedures. In certain areas still other mix design and stability test equipment may be required. The laboratory should be equipped for the locally prevalent practice.

10.4 *General Equipment*—Drying oven, thermometers, hot plates, glassware, scales, balances, mixing bowls, pans, miscellaneous laboratory tools, and special equipment which may be required for testing either bituminous materials or bituminous mixes to specifications under which the agency is operating or offers to operate.

10.5 *Batch and Mixing Plant Equipment*—Equipment needed in the bituminous mixing plant will, in general, be identical with that listed for the laboratory testing of bituminous mixes and aggregates for bitumen content and gradation. ASTM Recommended Practice D 290, Bituminous Mixing Plant Inspection,[7] provides a detailed discussion of services normally required.

10.6 *Field Inspection Equipment (at Job Site)*—Inasmuch as service at the project is usually confined to inspection of spreading and rolling of pavement and seldom involves any actual testing, little if any equipment is normally required at this point. The inspector should be provided with thermometers of appropriate ranges, steel rule, 100-ft tape, and, where required, facilities for removing samples for density tests and for checking thickness of pavement.

[13] Discontinued—see *1979 Annual Book of ASTM Standards*, Part 15.

REFERENCE MATERIAL

11. Reference Material

11.1 Appropriate references, relevant to the construction being inspected, including project plans and specifications, shall be readily available to the technicians at all times. The following are particularly essential:

11.1.1 Applicable parts of *Annual Book of ASTM Standards.*

11.1.2 *ACI Manual of Concrete Inspection,*

11.1.3 *Bureau of Reclamation Concrete Manual,*

11.1.4 *AISC Manual of Steel Construction,*

11.1.5 Applicable building codes,

11.1.6 *ASME Welding Code,*

11.1.7 *AWS Code for Welding in Building Construction.*

11.1.8 AWS Specifications for Welded Highway and Railway Bridges,

11.1.9 AWS Recommended Practice for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Construction,

11.1.10 AWS Special Ruling, Gas Metal Arc Welding with Carbon Dioxide Shielding,

11.1.11 CRSI Recommended Practice for Placing Reinforcing Bars, and

11.1.12 AASHTO standards.