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ACI MANUAL OF CONCRETE PRACTICE

PART 1—1980

Part 1 contains current committee reports and standards concerned with:

Materials and General Properties of Concrete

New editions of each part of the *ACI Manual of Concrete Practice* are issued annually and include the latest ACI standards and committee reports.

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The reports and standards herein were the latest approved versions at the time the contents of this edition were established. The content of each report or standard is subject to periodic review and to revision whenever the developments in concrete design and construction warrant a change. Since this is a continuing process, some reports or standards in this volume may have been superseded in the interim since publication. Inquiries concerning revisions or additional material in a subject area are welcome and should be directed to Institute headquarters.

Most standards and committee reports contained in this volume are also available as separate booklets from ACI headquarters. Prices supplied on request.

The American Concrete Institute publishes material on all phases of concrete technology. Much of the material can provide additional or background information on the reports and standards in this volume. A catalog is available.

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Contents

Materials and General Properties of Concrete

The AMERICAN CONCRETE INSTITUTE

is a nonprofit, nonpartisan organization of engineers, architects, scientists, constructors, and individuals associated in their technical interest with the field of concrete and dedicated to public service. The purpose of the Institute is to further engineering and technical education, scientific investigation and research, and development of standards for the design and construction of concrete structures. Members of the Institute are involved in gathering, correlating, and disseminating information for the improvement of the design, construction, manufacture, use, and maintenance of concrete products and structures. The Institute and its members also promote improved technology, technical competence, and good design and construction practices.

Since 1905 the objectives of the Institute have been achieved by a combined membership effort. Individually and through committees, and with the cooperation of many public and private agencies, members have correlated the results of research, from both field and laboratory, and of practices in design, construction, and manufacture.

The work of the Institute is made available to the engineering profession through seminars, workshops, chapter functions, and publications. The Institute publishes three periodicals: the ACI JOURNAL, *Concrete Abstracts,* and *Concrete International: Design & Construction.* The Institute also has an extensive nonperiodical publications program which includes committee reports, standards, symposia, manuals, design handbooks, monographs, education bulletins, bibliographies, and the *ACI Manual of Concrete Practice.*

Some of the most recent of these publications are:

SP-2	ACI Manual of Concrete Inspection
SP-17A(78)	Design Handbook in Accordance with the Strength Design Method of ACI 318-77: Volume 2—Columns
SP-61	Ferrocement—Materials and Applications
B-13	Concrete Core Tests
M-10	Concrete Box Girder Bridges
E2-78	Reinforcing Steel—Its Use and Benefits in Concrete

ACI MANUAL OF CONCRETE PRACTICE—1980

The *ACI Manual of Concrete Practice* is a five part compilation of current ACI standards and committee reports.

Part 1—Materials and General Properties of Concrete
Part 2—Construction Practices and Inspection
Pavements
Part 3—Use of Concrete in Buildings—Design, Specifications, and Related Topics
Part 4—Bridges, Substructures, Sanitary, and Other Special Structures
Structural Properties
Part 5—Masonry
Precast Concrete
Special Processes

Some of the most important work of the Institute is performed by its technical committees which prepare the committee reports and standards contained in the *Manual.* Technical committees of the Institute are organized into the following five groups with regard to their function: 100—Research and Administration, 200—Materials and Properties of Concrete, 300—Design and Construction, 400—Structural Analysis, and 500—Special Products and Special Processes. Committees are assigned a number which indicates its group or general area of responsibility.

Each standard of the Institute bears a hyphenated number to identify it. The first three digits identify the committee orginating the standard and the last two digits identify the year it was adopted. Thus standard ACI 214-77 was prepared by Committee 214 and was adopted as a standard in the year 1977.

Committee reports are also identified by a hyphenated number with the addition of an "R" to indicate a report rather than a standard. For committee reports the last two digits refer either to the year of original publication or in a few cases the year of adoption of a related standard.

The following consolidated list contains the titles of all committee reports and standards found in the *ACI Manual of Concrete Practice*—1980. Reports and standards are listed numerically and the location in the *Manual* follows the title.

100—Research and Administration

This group contains all research and administrative committees governed by TAC including any committees not logically placed in other subdivisions.

200—Materials and Properties of Concrete

This group contains committees whose major concern is materials in concrete and properties of concrete.

300—Design and Construction Practices

This group contains committees whose major concern is design and construction practices.

ACI **301-72, Revised 1975** Specifications for Structural Concrete for Buildings, **Part 3**

ACI **302-69** Recommended Practice for Concrete Floor and Slab Construction, **Part 3**

ACI **303R-74** Guide to Cast-in-Place Architectural Concrete Practice, **Part 3**

ACI **304-73, Reaffirmed 1978** Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete, **Part 2**

ACI **304.1R-69** Preplaced Aggregate Concrete for Structural and Mass Concrete, **Part 2**

ACI **304.2R-71** Placing Concrete by Pumping Methods, **Part 2**

ACI **304.3R-75** High Density Concrete: Measuring, Mixing, Transporting, and Placing, **Part 2**

ACI **304.4R-75** Placing Concrete With Belt Conveyors, **Part 2**

ACI **305R-77** Hot Weather Concreting, **Part 2**

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ACI **307-79** Specification for the Design and Construction of Reinforced Concrete Chimneys, **Part 4**

ACI **308-71, Reaffirmed 1978** Recommended Practice for Curing Concrete, **Part 2**

ACI **309-72, Reaffirmed 1978** Recommended Practice for Consolidation of Concrete, **Part 2**

ACI **311-75** Recommended Practice for Concrete Inspection, **Part 2**

ACI **311.1R-75** ACI Manual of Concrete Inspection (Synopsis only), **Part 2**

ACI **311.2R-63** Training Courses for Concrete Inspectors, **Part 2**

ACI **311.3R-75** Guide for Certification of Nuclear Concrete Inspection and Testing Personnel, **Part 2**

ACI **313-77** Recommended Practice for Design and Construction of Concrete Bins, Silos, and Bunkers for Storing Granular Materials, **Part 4**

ACI **313R-77** Commentary on Recommended Practice for Design and Construction of Concrete Bins, Silos, and Bunkers for Storing Granular Materials (ACI 313-77), **Part 4**

ACI **315-74, Revised 1978** Manual of Standard Practice for Detailing Reinforced Concrete Structures (Synopsis only), **Parts 3 and 4**

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ACI **325.3R-68** Guide for Design of Foundations and Shoulders for Concrete Pavements, **Part 2**

ACI **325.4R-72** A Design Procedure for Continuously Reinforced Concrete Pavements for Highways, **Part 2**

ACI **334.1R-74** Concrete Shell Structures—Practice and Commentary, **Part 4**

ACI **334.2R-77** Reinforced Concrete Cooling Tower Shells—Practice and Commentary, **Part 4**

ACI **336.1-79** Standard Specification for the Construction of End Bearing Drilled Piers, **Part 4**

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500—Special Products and Special Processes

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ACI 503.2-79 Standard Specification for Bonding Plastic Concrete to Hardened Concrete with a Multi-Component Epoxy Adhesive, **Part 5**

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MATERIALS AND GENERAL PROPERTIES OF CONCRETE

ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction, and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be incorporated directly into the Project Documents.

ACI 116R-78

Cement and Concrete Terminology

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Foreword

The first edition of the ACI glossary was the result of ten years of work by Committee 116. The second edition, which is based on the first edition, is the product of approximately six years of effort.

For the drafting of the second edition, the committee membership was divided into eight task groups, each of which was responsible for the preparation of an individual section. These sections were subsequently combined to form a base document for study and comment by the full committee. The comments resulting from this and other reviews, both within and without the committee, were considered by an Editorial Subcommittee composed of J. J. Shideler (Chairman), R. D. Gaynor, J. R. Libby, Bryant Mather, and R. C. Mielenz.

Committee 116 recognizes that despite meticulous attention to detail, the listing may not be complete and that some definitions may be at variance with some commonly accepted meanings. Users of the glossary are invited to submit suggestions for changes and additions to ACI Headquarters for consideration in preparing for future editions.

The Committee is aware that some of the definitions included may seem entirely self-evident to an expert in the concrete field. This occurs because no term has been discarded if there was reason to believe it would appear to be technical in nature to a casual reader of the ACI literature.

The invaluable contributions of the past-chairmen of Committee 116, R. C. Mielenz, D. L. Bloem, W. H. Price, and R. E. Davis, Jr., and of Bryant Mather as past-chairman of the Editorial Subcommittee, are gratefully acknowledged.

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A

Abrams' law—A rule stating that with given concrete materials and conditions of test the ratio of the amount of water to the amount of the cement in the mixture determines the strength of the concrete provided the mixture is of a workable consistency. (See also **Water-cement ratio.**)

Abrasion resistance—Ability of a surface to resist being worn away by rubbing and friction.

Absolute specific gravity—Ratio of the mass of a given volume of a solid or liquid, referred to a vacuum, at a stated temperature to the mass, referred to a vacuum, of an equal volume of gas-free distilled water at a stated temperature. (See also **Specific gravity.**)

Absolute volume (of ingredients of concrete or mortar)—The displacement volume of an ingredient of concrete or mortar; in the case of solids, the volume of the particles themselves, including their permeable and impermeable voids but excluding space between particles; in the case of fluids, the cubic content which they occupy.

Absorbed moisture—Moisture that has entered a solid material by absorption and has physical properties not substantially different from ordinary water at the same temperature and pressure. (See also **Absorption.**)

Absorption—The process by which a liquid is drawn into and tends to fill permeable pores in a porous solid body; also the increase in weight of a porous solid body resulting from the penetration of a liquid into its permeable pores. (See also **Absorbed moisture.**)

Abutment—In bridges, the end structure (usually of concrete) which supports the beams, girders, and deck of the bridge, or combinations thereof, and sometimes retains the earthen bank, or supports the end of the approach pavement slab; in prestressing, the structure against which the tendons are stressed in producing pretensioned precast members or post-tensioned pavement; in dams, the side of the gorge or bank of the stream against which a dam abuts.

Acceleration—Increase in velocity or in rate of change, especially the quickening of the natural progress of a process, such as hardening, setting, or strength development of concrete. (See also **Accelerator.**)

Accelerator—A substance which, when added to concrete, mortar, or grout, increases the rate of hydration of the hydraulic cement, shortens the time of setting, or increases the rate of hardening of strength development, or both. (See also **Acceleration.**)

Accessories—Those items other than frames, braces, or post shores used to facilitate the construction of scaffold and shoring.

Accidental air—See **Entrapped air**.

Acrylic resin—One of a group of thermoplastic resins formed by polymerizing the esters or amides of acrylic acid; used in concrete construction as a bonding agent or surface sealer.

Addition—A material that is interground or blended in limited amounts into a hydraulic cement during manufacture either as a "processing addition" to aid in manufacturing and handling the cement or as a "functional addition" to modify the use properties of the finished product.

Additive—A term frequently (but improperly) used as a synonym for addition or admixture.

Adhesives—The group of materials used to join or bond similar or dissimilar materials; for example, in concrete work, the epoxy resins.

Adiabatic—A condition in which heat neither enters nor leaves a system.

Adiabatic curing—The maintenance of adiabatic conditions in concrete or mortar during the curing period.

Adjustment screw—A leveling device or jack composed of a threaded screw and an adjusting handle used for the vertical adjustment of shoring and formwork.

Admixture—A material other than water, aggregates, and hydraulic cement, used as an ingredient of concrete or mortar, and added to the concrete immediately before or during its mixing.

Adobe—Unburnt brick dried in the sun.

Adsorbed water—Water held on surfaces of a material by electrochemical forces and having physical properties substantially different from those of absorbed water or chemically combined water at the same temperature and pressure. (See also **Adsorption.**)

Adsorption—Development at the surface of a liquid or solid of a higher concentration of a substance than exists in the bulk of the medium; especially formation of one or more layers of molecules of gases, of dissolved substances, or of liquids at the surface of a solid, such as cement, cement paste, or aggregate, or of air-entraining agents at the air-water interfaces; also the process by which a substance is adsorbed. (See also **Adsorbed water.**)

Advancing slope grouting—A method of grouting by which the front of a mass of grout is caused to move horizontally through preplaced aggregate by use of a suitable grout injection sequence.

Advancing slope method—A method of placing concrete as in tunnel linings in which the face of the fresh concrete is not vertical and moves forward as concrete is placed.

Aerated concrete—See **Cellular concrete**.

A/F ratio—The molar or weight ratio of aluminum oxide (Al_2O_3) to iron oxide (Fe_2O_3), as in portland cement.

Afwillite—A mineral with composition $3CaO \cdot 2SiO_2 \cdot 3H_2O$ occurring naturally in South Africa, Northern Ireland, and California, and artificially in some hydrated portland cement mixtures.

Agglomeration—Gathering into a ball or mass.

Aggregate—Granular material, such as sand, gravel, crushed stone, and iron blast-furnace slag, used with a cementing medium to form a hydraulic-cement, concrete or mortar. (See also **Aggregate, heavyweight** and **Aggregate, lightweight.**)

Aggregate, coarse—See **Coarse aggregate.**

Aggregate, fine—See **Fine aggregate.**

Aggregate, gap-graded—See **Gap-graded aggregate.**

Aggregate, heavyweight—Aggregate of high specific gravity such as barite, magnetite, limonite, ilmenite, iron, or steel used to produce heavy concrete.

Aggregate, lightweight—Aggregate of low specific gravity, such as expanded or sintered clay, shale, slate, diatomaceous shale, perlite, vermiculite, or slag; natural pumice, scoria, volcanic cinders, tuff, and diatomite; sintered fly ash or industrial cinders; used to produce lightweight concrete.

Aggregate, maximum size—See **Maximum size of aggregate.**

Aggregate, nominal maximum size—See **Nominal maximum size of aggregate.**

Aggregate blending—The process of intermixing two or more aggregates to produce a different set of properties; generally, but not exclusively, to improve grading.

Aggregate-cement ratio—See Cement-aggregate ratio.

Aggregate gradation—See **Grading**.

Aggregate interlock—The projection of aggregate particles or portions of aggregate particles from one side of a joint or crack in concrete into recesses in the other side of such joint or crack so as to effect load transfer in compression and shear, and maintain mutual alignment.

Agitating speed—The rate of rotation of the drum or blades of a truck mixer or other device when used for agitation of mixed concrete.

Agitating truck—A vehicle carrying a drum in which freshly mixed concrete can be conveyed from the point of mixing to that of placing, the drum being rotated continuously so as to agitate the contents; designated "agitating lorry" in United Kingdom.

Agitation—

1. The process of providing gentle motion in mixed concrete just sufficient to prevent segregation or loss of plasticity.

2. The mixing and homogenization of slurries or finely ground powders by air or mechanical means.

See also **Agitator**.

Agitator—A device for maintaining plasticity and preventing segregation of mixed concrete by agitation. (See also **Agitation**.)

Air, entrained—See **Entrained air**.

Air blow pipe—Air jet used in shotcrete gunning to remove rebound or other loose material from the work area.

Air-blown mortar—See **Shotcrete**.

Air content—The volume of air voids in cement paste, mortar, or concrete, exclusive of pore space in aggregate particles, usually expressed as a percentage of total volume of the paste, mortar, or concrete.

Air-cooled blast-furnace slag—See **Blast-furnace slag**.

Air-entraining—The capability of a material or process to develop a system of minute bubbles of air in cement paste, mortar, or concrete during mixing. (See also **Air entrainment**.)

Air entraining agent—An addition for hydraulic cement or an admixture for concrete or mortar which causes entrained air to be incorporated in the concrete or mortar during mixing, usually to increase its workability and frost resistance. (See also **Entrained air**.) (small bubbles)

Air-entraining hydraulic cement—Hydraulic cement containing an air-entraining addition in such amount as to cause the product to entrain air in mortar within specified limits.

Air entrainment—The occlusion of air in the form of minute bubbles (generally smaller than 1 mm) during the mixing of concrete or mortar. (See also **Air entraining** and **Entrained air**.)

Air lift—Equipment whereby slurry or dry powder is lifted through pipes by means of compressed air.

Air meter—A device for measuring the air content of concrete and mortar.

Air permeability test—A procedure for measuring the fineness of powdered materials such as portland cement.

Air ring—Perforated manifold in nozzle of wet-mix shotcrete equipment through which high pressure air is introduced into the material flow.

Air separator—An upright cylindrical-conical apparatus, with internal rotating blades, which separates various size fractions of ground materials pneumatically; fine particles are discharged as product; oversize is returned to the mill as tailing.

Air void—A space in cement paste, mortar, or concrete filled with air; an entrapped air void is characteristically 1 mm or more in size and irregular in shape; an entrained air void is typically between 10 and 1000μm in diameter and spherical or nearly so.

Air-water jet—A high-velocity jet of air and water mixed at the nozzle, used in clean-up of surfaces of rock or concrete such as horizontal construction joints.

Alabaster—A massive densely crystalline, softly textured form of practically pure gypsum.

Alignment wire—See **Ground wire**.

Alite—A name used by Tornebohm (1897) to identify tricalcium silicate including small amounts of MgO, Al_2O_3, Fe_2O_3, and other oxides; a principal constituent of portland-cement clinker. (See also **Belite, Celite,** and **Felite**.)

Alkali—Salts of alkali metals, principally sodium and potassium; specifically sodium and potassium occurring in constituents of concrete or mortar, usually expressed in chemical analyses as the oxides Na_2O and K_2O. (See also **Cement, low alkali**.)

Alkali-aggregate reaction—Chemical reaction in mortar or concrete between alkalies (sodium and potassium) from portland cement or other sources and certain constituents of some aggregates; under certain conditions, deleterious expansion of the concrete or mortar may result.

Alkali-carbonate rock reaction—The reaction between the alkalies (sodium and potassium) in portland cement and certain carbonate rocks, particularly calcitic dolomite and dolomitic limestones, present in some aggregates; the products of the reaction may cause abnormal expansion and cracking of concrete in service.

Alkali reactivity (of aggregate)—Susceptibility of aggregate to alkali-aggregate reaction.

Alkali-silica reaction—The reaction between the alkalies (sodium and potassium) in portland cement and certain siliceous rocks or minerals, such as opaline chert and acidic volcanic glass, present in some aggregates; the products of the reaction may cause abnormal expansion and cracking of concrete in service.

Alkyl aryl sulfonate—Synthetic detergent from petroleum fractions.

Allowable load—The ultimate load divided by factor of safety.

Allowable stress—Maximum permissible stress used in design of members of a structure and based on a factor of safety against rupture or yielding of any type.

Alternate lane construction—A method of constructing concrete roads, runways, or other paved areas, in which alternate lanes are placed and allowed to harden before the remaining intermediate lanes are placed.

Alumina—Aluminum oxide (Al_2O_3).

Aluminate cement—See Calcium-aluminate cement.

Aluminate concrete—Concrete made with calcium-aluminate cement; used primarily where high-early-strength or refractory or corrosion-resistant concrete is required.

Amount of mixing—The designation of extent of mixer action employed in combining the ingredients for concrete or mortar; in the case of stationary mixers, the mixing time; in the case of truck mixers, the number of revolutions of the drum or blades at mixing speed after the intermingling of the cement with water and aggregates. (See also **Mixing time**.)

Amplitude—The maximum displacement from the mean position in connection with vibration.

Anchor—In prestressed concrete, to lock the stressed tendon in position so that it will retain its stressed condition; in precast concrete construction, to attach the precast units to the building frame; in slabs on grade or walls, to fasten to rock or adjacent structures to prevent movement of the slab or wall with respect to the foundation, adjacent structure or rock. (See also **Form anchor**.)

Anchor bolt—A metal bolt or stud, headed or threaded, either cast in place, grouted in place, or drilled into finished concrete, used to hold various structural members or embedments in the concrete, and to resist shear, tension, and vibration loadings from various sources such as wind, machine vibration, etc.; known also as a **Hold-down bolt** or a **Foundation bolt**.

Anchorage—In post-tensioning, a device used to anchor the tendon to the concrete member; in pretensioning, a device used to anchor the tendon during hardening of the concrete; in precast concrete construction, the devices for attaching precast units to the building frame; in slab or wall construction, the device used to anchor the slab or wall to the foundation, rock, or adjacent structure.

Anchorage bond stress—The bar forces divided by the product of the bar perimeter or perimeters and the embedment length.

Anchorage deformation or **slip**—The loss of elongation or stress in the tendons of prestressed concrete due to the deformation of the anchorage or slippage of the tendons in the anchorage device when the prestressing force is transferred from the jack to the anchorage device.

Anchorage device—See **Anchorage.**

Anchorage loss—See **Anchorage deformation** or **slip.**

Anchorage zone—In post-tensioning, the region adjacent to the anchorage subjected to secondary stresses resulting from the distribution of the prestressing force; in pretensioning, the region in which the transfer bond stresses are developed.

Angle float—A finishing tool having a surface bent to form a right angle; used to finish re-entrant angles.

Angle of repose—The angle between the horizontal and the natural slope of loose material below which the material will not slide.

Angular aggregate—Aggregate, the particles of which possess well-defined edges formed at the intersection of roughly planar faces.

Anhydrite—A mineral, anhydrous calcium sulfate ($CaSO_4$); gypsum from which the water of crystallization has been removed, usually by heating above 325 F (160 C); natural anhydrite is less reactive than that obtained by calcination of gypsum.

Apparent specific gravity—See **Specific gravity**.

Architect-Engineer or **Engineer-Architect**—The architect, engineer, architectural firm, engineering firm, or architectural and engineering firm, issuing project drawings and specifications, or administering the work under contract specifications and drawings, or both.

Architectural concrete—Concrete which will be permanently exposed to view and which therefore requires special care in selection of the concrete materials, forming, placing, and finishing to obtain the desired architectural appearance.

Arc spectrography—Spectographic identification of elements in a sample of material, heated to volatilization in an electric arc or spark.

Area of steel—The cross-sectional area of the reinforcing bars in or for a given concrete cross-section. (See also **Effective area of reinforcement.**)

Arenaceous—Composed primarily of sand; sandy.

Argillaceous—Composed primarily of clay or shale; clayey.

Arrissing tool—A tool similar to a float, but having a form suitable for rounding an edge of freshly placed concrete.

Asbestos-cement products—Products made from rigid material composed essentially of asbestos fiber and portland cement, used in a wide range of forms in the building industry.

Ashlar—Masonry composed of squared stones; one pattern of masonry construction.

Asphalt concrete—See **Concrete, asphalt.**

Atmospheric-pressure steam curing—Steam curing of concrete products or cement at atmospheric pressure, usually at maximum ambient temperature between 100–200 F (40–95 C).

Atterberg limits—Arbitrary water contents (shrinkage limit, plastic limit, liquid limit) determined by standard tests, which define the boundaries between the different states of consistency of plastic soils.

Atterberg test—A method for determining the plasticity of soils.

Autoclave—A pressure vessel in which an environment of steam at high pressure may be produced; used in the curing of concrete products and in the testing of hydraulic cement.

Autoclave curing—Steam curing of concrete products, sand-lime brick, asbestos-cement products, hydrous calcium silicate insulation products, or cement in an autoclave at maximum ambient temperatures generally between 340-420 F (170-215 C).

Autoclave cycle—The time interval between the start of the temperature-rise period and the end of the blowdown period; also, a schedule of the time and temperature-pressure conditions of periods which make up the cycle.

Autoclaved—See **Autoclave curing.**

Autoclaving—See **Autoclave curing.**

Autogenous healing—A natural process of closing and filling of cracks in concrete or mortar when the concrete or mortar is kept damp.

Autogenous volume change—Change in volume produced by continued hydration of cement exclusive of effects of external forces or change of water content or temperature.

Automatic batcher—See **Batcher**.

Auxiliary reinforcement—In a prestressed member, any reinforcement in addition to that participating in the prestressing function.

Average bond stress—The force in a bar divided by the product of its perimeter and its embedded length.

Axle load—The portion of the gross weight of a vehicle transmitted to a structure or a roadway through wheels supporting a given axle.

Axle steel—Steel from carbon-steel axles for railroad cars.

Axle steel reinforcement—Plain or deformed reinforcing bars rolled from axle steel.

B

b/b_0—See **Coarse aggregate factor.**

Backfill concrete—Non-structural concrete used to correct over-excavation, or fill excavated pockets in rock, or to prepare a surface to receive structural concrete.

Back form—See **Top form.**

Back plastering—Plaster applied to one face of a lath system following application and subsequent hardening of plaster applied to the opposite face.

Back stay—See **Brace.**

Bacterial corrosion—The destruction of a material by chemical processes brought about by the activity of certain bacteria which may produce substances such as hydrogen sulfide, ammonia, and sulfuric acid.

Bag (of cement; also **Sack)**—A quantity of portland cement: 94 lb in the United States, and 50 kg in most other countries; for other kinds of cement a quantity indicated on the bag (obsolete).

Balanced load—Load capacity at simultaneous crushing of concrete and yielding of tension steel.

Balanced moment—Moment capacity at simultaneous crushing of concrete and yielding of tension steel.

Balanced reinforcement—An amount and distribution of reinforcement in a flexural member such that in working stress design the allowable tensile stress in the steel and the allowable compressive stress in the concrete are attained simultaneously; or such that in strength design the tensile reinforcement reaches its specified yield strength simultaneously with the concrete in compression reaching its assumed ultimate strain of 0.003.

Ball mill—Horizontal, cylindrical, rotating mill charged with large grinding media.

Ball test—A test to determine the consistency of freshly mixed concrete by measuring the depth of penetration of a cylindrical metal weight with a hemispherical bottom.

Band—Small bars or wire encircling the main reinforcement in a member to form a peripheral tie; group of bars distributed in a slab, or wall, or footing.

Band iron—Thin metal strap used as form tie, hanger, etc.

Bar—A member used to reinforce concrete.

Bar bender—A tradesman who cuts and bends steel reinforcement; or a machine for bending reinforcement.

Bar chair—An individual supporting device used to support or hold reinforcing bars in proper position to prevent displacement before or during concreting.

Bar, deformed—See **Deformed bar.**

Bar mat—An assembly of steel reinforcement composed of two or more layers of bars placed at angles to each other and secured together by welding or ties.

Bar spacing—The distance between parallel reinforcing bars, measured center to center of the bars perpendicular to their longitudinal axes.

Bar support—A rigid device used to support or hold reinforcing bars in proper position to prevent displacement before or during concreting.

Barite—A mineral, barium sulfate ($BaSO_4$), used in pure or impure form as concrete aggregate primarily for the construction of high-density radiation shielding concrete; designated "barytes" in United Kingdom.

Barrage—A low dam erected to control the level of a stream.

Barrel (of cement)—A quantity of portland cement; 376 lb (4 bags) in the United States (obsolete); also wood or metal container formerly used for shipping cement.

Barrel-vault roof—A thin concrete roof taking the form of a part of a cylinder.

Base—A subfloor slab or "working mat," either previously placed and hardened or freshly placed, on which floor topping is placed in a later operation; also the underlying stratum on which a concrete slab, such as a pavement, is placed. (See also **Subbase.**)

Base bead—See **Base screed.**

Base coat—Any plaster coat or coats applied prior to application of the finished coat.

Base course—A layer of specified selected material of planned thickness constructed on the subgrade or subbase of a pavement to serve one or more functions such as distributing loads, providing drainage, or minimizing frost action; also the lowest course of masonry in a wall or pier.

Base plate—A plate of metal or other approved material formerly placed under pavement joints and the adjacent slab ends to prevent the infiltration of soil and moisture from the sides or bottom of the joint opening; also a device used to distribute vertical loads as for building columns or machinery.

Base screed—A preformed metal screed with perforated or expanded flanges to provide a ground for plaster and to separate areas of dissimilar materials.

Basket—See **Load-transfer assembly.**

Bassanite—Calcium sulfate hemihydrate, $2CaSO_4 \cdot H_2O$. (See also **Hemihydrate** and **Plaster of Paris.**)

Bat—A broken, burned brick or shape.

Batch—Quantity of concrete or mortar mixed at one time.

Batch box—Container of known volume used to measure constituents of a batch of concrete or mortar in proper proportions.

Batch mixer—A machine which mixes batches of concrete or mortar in contrast to a continuous mixer.

Batch plant—An operating installation of equipment including batchers and mixers as required for batching or for batching and mixing concrete materials; also called mixing plant when mixing equipment is included.

Batch weights—The weights of the various materials (cement, water, the several sizes of aggregate, and admixtures if used) of which a batch of concrete is composed.

Batched water—The mixing water added by a batcher to a concrete or mortar mixture before or during the initial stages of mixing.

Batcher—A device for measuring ingredients for a batch of concrete.

1. **Manual batcher**—A batcher equipped with gates or valves which are operated manually, with or without supplementary power from pneumatic, hydraulic, or electrical machinery, the accuracy of the weighing operation being dependent on the operator's observation of the scale.

2. **Semiautomatic batcher**—A batcher equipped with gates or valves which are separately opened manually to allow the material to be weighed but which are closed automatically when the designated weight of each material has been reached.

3. **Automatic batcher**—A batcher equipped with gates or valves which, when actuated by a single starter switch, will open automatically at the start of the weighing operation of each material and close automatically when the designated weight of each material has been

reached, interlocked in such a manner that: (a) the charging mechanism cannot be opened until the scale has returned to zero; (b) the charging mechanism cannot be opened if the discharge mechanism is open; (c) the discharge mechanism cannot be opened if the charging mechanism is open; (d) the discharge mechanism cannot be opened until the designated weight has been reached within the allowable tolerance; and (e) if different kinds of aggregates or different kinds of cements are weighed cumulatively in a single batcher, interlocked sequential controls are provided.

√ **Batching**—Weighing or volumetrically measuring and introducing into the mixer the ingredients for a batch of concrete or mortar.

Batten (also **Batten strip**)—A narrow strip of wood placed over the vertical joint of sheathing or paneling, or used to hold several boards together. (See also **Cleat**.)

Batter—Inclination from the vertical or horizontal.

Batter boards—Pairs of horizontal boards nailed to wood stakes adjoining an excavation, used as a guide to elevations and to outline the building.

Batter pile—A pile which is installed at an angle to the vertical; a raking pile.

Bauxite—A rock composed principally of hydrous aluminum oxides; the principal ore of aluminum, and a raw material for manufacture of calcium aluminate cement.

Bay—The space between two adjacent piers or mullions or between two adjacent lines of columns; a small, well-defined area of concrete laid at one time in the course of placing large areas such as floors, pavements, or runways.

Beam—A structural member subjected primarily to flexure; also the gradated horizontal bar of a weighing scale on which the balancing poises ride. (See also **Girder, Girt, Joist, Ledger, Purlin, Spandrel beam,** and **Stringer**.)

Beam-and-slab floor—A reinforced concrete floor system in which the floor slab is supported by beams of reinforced concrete.

Beam bottom—Soffit or bottom form for a beam.

Beam clamp—Any of various types of tying or fastening units used to hold the sides of beam forms.

Beam-column—A structural member which is subjected to forces producing significant amounts of both bending and compression simultaneously.

Beam form—A retainer or mold so erected as to give the necessary shape, support, and finish to a concrete beam.

Beam hanger—A wire, strap, or other hardware device that supports formwork from structural members.

Beam pocket—Opening left in a vertical member in which a beam is to rest; also an opening in the column or girder form where forms for an intersecting beam will be framed.

Beam saddle—See **Beam hanger**.

Beam side—Vertical side panels or parts of a beam form.

Beam test—A method of measuring the flexural strength (modulus of rupture) of concrete by testing a standard unreinforced beam.

Bearing capacity—The maximum unit pressure which a soil or other material will withstand without failure or without settlement to an amount detrimental to the integrity or the function of the structure.

Bearing stratum—The soil or rock stratum on which a footing or mat bears or which carries the load transferred to it by a pile, pier, caisson, or similar deep foundation unit.

Belite—A name used by Tornebohm (1897) to identify one form of the constituent of portland cement clinker now known when pure as dicalcium silicate ($2CaO \cdot SiO_2$). (See also **Alite, Celite** and **Felite**.)

Bench—See **Pretensioning bed**.

Bending moment—The bending effect at any section of a structural element; it is equal to the algebraic sum of all moments to the right or left of the section.

Bending moment diagram—A graphical representation of the variation of bending moment along the length of the member for a given stationary system of loads.

Bending schedule—A list of reinforcement prepared by the designer or detailer of a reinforced concrete structure, showing the shapes and dimensions of every bar and the number of bars required.

Beneficiation—Improvement of the chemical or physical properties of a raw material or intermediate product by the removal of undesirable components or impurities.

Bent—Two-dimensional frame which is self-supporting within these dimensions, having at least two legs and usually placed at right angles to the length of the structure which it supports.

Bent bar—A reinforcing bar bent to a prescribed shape such as a truss bar, straight bar with hook, stirrup, or column tie.

Bentonite—A clay composed principally of minerals of the montmorillonoid group, characterized by high adsorption and very large volume change with wetting or drying.

Berliner—A type of terrazzo topping using small and large pieces of marble paving, usually with a standard terrazzo matrix between pieces.

Billet steel—Steel, either reduced directly from ingots or continuously cast, made from properly identified heats of open-hearth, basic oxygen, or electric furnace steel, or lots of acid Bessemer steel and conforming to specified limits on chemical composition.

Binders—Cementing materials, either hydrated cements or products of cement or lime and reactive siliceous materials; the kinds of cement and curing conditions govern the general kind of binder formed; also materials such as asphalt, resins, and other materials forming the matrix of concretes, mortars, and sanded grouts.

Biological shielding—Shielding provided to attenuate or absorb nuclear radiation, such as neutron, proton, alpha, beta, and gamma particles; the shielding is provided mainly by the density of the concrete, except that in the case of neutrons the attenuation is achieved by compounds of some of the lighter elements (e.g., hydrogen and boron). (See also **Shielding concrete**.)

Blaine apparatus—Air-permeability apparatus for measuring the surface area of a finely ground cement, raw material, or other product.

Blaine fineness—The fineness of powdered materials such as cement and pozzolans, expressed as surface area usually in square centimeters per gram, determined by the Blaine apparatus. (See also **Specific surface**.)

Blaine test—A method for determining the fineness of cement or other fine material on the basis of the permeability to air of a sample prepared under specified conditions.

Blast-furnace slag—The nonmetallic product, consisting essentially of silicates and aluminosilicates of calcium and other bases, that is developed in a molten condition simultaneously with iron in a blast furnace.

1. **Air-cooled blast-furnace slag** is the material resulting from solidification of molten blast-furnace slag under atmospheric conditions; subsequent cooling may be accelerated by application of water to the solidified surface.

2. **Expanded blast-furnace slag** is the lightweight, cellular material obtained by controlled processing of molten blast-furnace slag with water, or water and other agents, such as steam or compressed air, or both.

3. **Granulated blast-furnace slag** is the glassy, granular material formed when molten blast-furnace slag is rapidly chilled, as by immersion in water.

Bleed—To undergo bleeding. (See **Bleeding.**)

Bleeding—The autogenous flow of mixing water within, or its emergence from newly placed concrete or mortar; caused by the settlement of the solid materials within the mass; also called water gain.

Bleeding capacity—The ratio of volume of water released by bleeding to the volume of paste or mortar.

Bleeding rate—The rate at which water is released from a paste or mortar by bleeding.

Blending cement—See **Cement, blended**.

Blinding—The application of a layer of weak concrete or other suitable material to reduce surface voids, or to provide a clean dry working surface; also the filling or plugging of the openings in a screen or sieve by the material being separated.

Blistering—The irregular raising of a thin layer at the surface of placed mortar or concrete during or soon after completion of the finishing operation, or in the case of pipe after spinning; also bulging of the finish plaster coat as it separates and draws away from the base coat.

Bloated—Swollen, as in certain lightweight aggregates as a result of processing.

Block—A concrete masonry unit, usually containing hollow cores; also a solid piece of wood or other material to fill spaces between formwork members.

Block beam—A flexural member composed of individual blocks which are joined together by prestressing.

Blockout—A space within a concrete structure under construction in which fresh concrete is not to be placed, called **Core** in United Kingdom.

Blowdown period—Time taken to reduce pressure in an autoclave from maximum to atmospheric.

Blowhole—See **Bug holes**.

Board butt joint—Shotcrete construction joint formed by sloping gunned surface to a 1-in. board laid flat.

Bolster, slab—Continuous wire bar support used to support bars in the bottom of slabs; top wire corrugated at one inch centers to hold bars in position.

Board butt joint—Shortcrete construction joint formed by sloping gunned surface to a 1-in. board laid flat.

Bolt sleeve—A tube surrounding a bolt in a concrete wall to prevent concrete from sticking to the bolt and acting as a spreader for the formwork.

Bond—Adhesion and grip of concrete or mortar to reinforcement or to other surfaces against which it is placed, including friction due to shrinkage and longitudinal shear in the concrete engaged by the bar deformations; the adhesion of cement paste to aggregate; adherence between plaster coats or between plaster and a substrata produced by adhesive or cohesive properties of plaster or supplemental materials; also in United Kingdom the arrangement of units in masonry and brickwork so that vertical joints are discontinuous.

Bond area—The area of interface between two elements across which adhesion develops or may develop, as between concrete and reinforcing steel.

Bond breaker—A material used to prevent adhesion of newly placed concrete and the substrate.

Bond length—See **Development length**.

Bond plaster—A specially formulated gypsum plaster designed as first coat application over monolithic concrete.

Bond prevention—Procedures whereby specific tendons in pretensioned construction are prevented from becoming bonded to the concrete for a predetermined distance from the ends of flexural members; measures taken to prevent adhesion of concrete or mortar to surfaces against which it is placed.

Bond strength—Resistance to separation of mortar and concrete from reinforcing steel and other materials with which it is in contact; a collective expression for all forces such as adhesion, friction due to shrinkage, and longitudinal shear in the concrete engaged by the bar deformations that resist separation.

Bond stress—The force of adhesion per unit area of contact between two bonded surfaces such as concrete and reinforcing steel or any other material such as foundation rock; shear stress at the surface of a reinforcing bar, preventing relative movement between the bar and the surrounding concrete.

Bonded member—A prestressed concrete member in which the tendons are bonded to the concrete either directly or through grouting.

Bonded post-tensioning—Post-tensioned construction in which the annular spaces around the tendons are grouted after stressing, thereby bonding the tendon to the concrete section.

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

Bonder (Header)—A masonry unit which ties two or more wythes (leaves) of a wall together by overlapping.

Bonding agent—A substance applied to a suitable substrate to create a bond between it and a succeeding layer as between a subsurface and a terrazzo topping or a succeeding plaster application.

Bonding layer—A layer of mortar, usually ⅛ to ½ in. (3 to 13 mm) thick, which is spread on a moist and prepared, hardened concrete surface prior to placing fresh concrete.

Bored pile—A concrete pile, with or without a casing, cast-in-place in a hole previously bored in soil or rock. (See also **Cast-in-place pile.**)

Boron frits—Clear, colorless, synthetic glass produced by fusion and quenching, containing boron. (See also **Boron-loaded concrete.**)

Boron-loaded concrete—High-density concrete including a boron-containing admixture or aggregate, such as mineral colemanite, boron frits, or boron metal alloys, to act as a neutron attenuator. (See also **Biological shielding** and **Shielding concrete.**)

Box out—To form an opening or pocket in concrete by a box-like form.

Brace—Any structural member used to support another; always designed for compression and sometimes for tension under special load conditions.

Bracing—Structural elements, which due to their ability to transmit direct stress, are provided to either prevent buckling of individual members subject to compression, to add rigidity to a structure as a whole, or to resist lateral loads. A member used to support, strengthen, or position another piece or portion of a framework.

Bracket—An overhanging member projecting from a wall or other body to support weight acting outside the wall, or similar piece to strengthen an angle.

Breccia—Rock composed of angular fragments of older rock cemented together.

Bredigite—A mineral, alpha prime dicalcium silicate ($2CaO \cdot SiO_2$), occurring naturally at Scawt Hill, Northern Ireland; and at Isle of Muck, Scotland; also in slags and portland cement.

Breeze—Usually clinker; also fine divided material from coke production.

Brick seat—Ledge on wall or footing to support a course of masonry.

Bridge deck—The slab or other structure forming the travel surface of a bridge.

Briquette (also **Briquet**)—A molded specimen of mortar with enlarged extremities and reduced center having a cross section of definite area, used for measurement of tensile strength.

Broadcast—To toss granular material, such as sand, over a horizontal surface so that a thin, uniform layer is obtained.

Broom finish—The surface texture obtained by stroking a broom over freshly placed concrete. (See also **Brushed surface**.)

Brown coat—The second coat in three-coat plaster application.

Brown out—To complete application of basecoat plaster.

Brown oxide—A brown mineral pigment having an iron oxide content between 28 and 95 percent. (See also **Limonite**.)

Brownmillerite—A ternary compound originally regarded as $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$ (C_4AF) occurring in portland and calcium aluminate cement; now used to refer to a series of solid solutions between $2CaO \cdot Fe_2O_3$ (C_2F) and $2CaO \cdot Al_2O_3$ (C_2A).

Brucite—A mineral having the composition magnesium hydroxide, $Mg(OH)_2$, and a specific crystal structure.

Brushed surface—A sandy texture obtained by brushing the surface of freshly placed or slightly hardened concrete with a stiff brush for architectural effect or, in pavements, to increase skid resistance. (See also **Broom finish**.)

Buck—Framing around an opening in a wall; a door buck encloses the opening in which a door is placed.

Buckling—Failure by lateral or torsional instability of a structural member, occurring with stresses below the yield or ultimate values.

Bug holes—Small regular or irregular cavities, usually not exceeding 15 mm in diameter, resulting from entrapment of air bubbles in the surface of formed concrete during placement and compaction.

Buggy—A two-wheeled or motor-driven cart usually rubber-tired, for transporting small quantities of concrete from hoppers or mixers to forms; sometimes called a concrete cart.

Building official—The official charged with administration and enforcement of the applicable building code, or his duly authorized representative.

Build-up—Gunning of shotcrete in successive layers to form a thicker mass; also the accumulation of residual hardened concrete in a mixer.

Bulk cement—Cement which is transported and delivered in bulk (usually in specially constructed vehicles) instead of in bags.

Bulk density—The weight of a material (including solid particles and any contained water) per unit volume including voids. (See also **Specific gravity**.)

Bulk loading—Loading of unbagged cement in containers, specially designed trucks, railroad cars, or ships.

Bulk specific gravity—See **Specific gravity**.

Bulkhead—A partition in formwork blocking fresh concrete from a section of the form or closing a section of the form, such as at a construction joint; a partition in a storage tank or bin, as for cement or aggregate.

Bulking—Increase in the bulk volume of a quantity of sand in a moist condition over the volume of the same quantity dry or completely inundated.

Bulking curve—Graph of change in volume of a quantity of sand due to change in moisture content.

Bulking factor—Ratio of the volume of moist sand to the volume of the sand when dry.

Bull float—A tool comprising a large, flat, rectangular piece of wood, aluminum, or magnesium usually 8 in. (20 cm) wide and 42 to 60 in. (100 to 150 cm) long, and a handle 4 to 16 ft (1 to 5 m) in length used to smooth unformed surfaces of freshly placed concrete.

Bundled bars—A group of not more than four parallel reinforcing bars in contact with each other, usually tied together.

Burlap—A coarse fabric of jute, hemp, or less commonly, flax, for use as a water-retaining covering in curing concrete surfaces; also called **Hessian**.

Bush-hammer—A hammer having a serrated face, as rows of pyramidal points used to roughen or dress a surface; to finish a concrete surface by application of a bush-hammer.

Bush-hammer finish—A finish on concrete obtained by means of a bush-hammer.

Butt joint—A plain square joint between two members.

Buttering—Process of spreading mortars on a brick or other masonry unit with a trowel; also the process by which the interior of a concrete mixer, transportation unit, or other item coming in contact with fresh concrete is provided with a mortar coating so that the fresh concrete coming in contact with it will not be depleted of mortar.

Buttress—A projecting structure to support a wall or building.

Butyl stearate—A colorless oleaginous, practically odorless material ($C_{17}H_{35}COOC_4H_9$) used as a dampproofer for concrete.

C

Cable—See **Tendon.**

Cage—A rigid assembly of reinforcement ready for placing in position.

Caisson pile—A cast-in-place pile made by driving a tube, excavating it, and filling the cavity with concrete.

Calcareous—Containing calcium carbonate or, less generally, containing the element calcium.

Calcine—To alter composition of physical state by heating below the temperature of fusion.

Calcite—A mineral having the composition calcium carbonate ($CaCO_3$) and a specific crystal structure; the principal constituent of limestone, chalk, and marble; used as a major constituent in the manufacture of portland cement.

Calcium—A silver-white metallic element of the alkaline-earth group occurring only in combination with other elements.

Calcium-aluminate cement—The product obtained by pulverizing clinker consisting essentially of hydraulic calcium aluminates resulting from fusing or sintering a suitably proportioned mixture of aluminous and calcareous materials, called **High-alumina cement** in United Kingdom.

Calcium chloride—A crystalline solid, $CaCl_2$; in various technical grades, used as a drying agent, as an accelerator of concrete, a deicing chemical, and for other purposes.

Calcium stearate—Product of the reaction of lime and stearic acid used as an integral water repellant in concrete.

Calcium-silicate brick—A concrete product made principally from sand and lime which is hardened by autoclave curing.

Calcium-silicate hydrate—Any of the various reaction products of calcium silicate and water, often produced by autoclave curing.

Caliche—Gravel, sand, and desert debris cemented by calcium carbonate or other salts.

California bearing ratio—The ratio of the force per unit area required to penetrate a soil mass with a 3 sq in. (19.4 sq cm) circular piston at the rate of 0.05 in. (1.27 mm) per min to the force required for corresponding penetration of a standard crushed-rock base material; the ratio is usually determined at 0.1 in. (2.5 mm) penetration.

Calorimeter—An instrument for measuring heat exchange during a chemical reaction such as the quantities of heat liberated by the combustion of a fuel or hydration of a cement.

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

Cant strip—See **Chamfer strip**.

Cap—A smooth, plane surface of suitable material bonded to the bearing surfaces of test specimens to insure uniform distribution of load during strength testing.

Cap cables—Short cables (tendons) introduced to prestress the zone of negative bending only.

Capacity—A measure of the rated volume of a particular concrete mixer or agitator, usually limited by specifications to a maximum percentage of total gross volume; also the output of concrete, aggregate, or other product per unit of time (as plant capacity or screen capacity); also load carrying limit of a structure.

Capacity reduction factor—See **Phi (ϕ) factor**.

Capillarity—The movement of a liquid in the interstices of soil or other porous material due to surface tension. (See also **Capillary flow**.)

Capillary flow—Flow of moisture through a capillary pore system, such as in concrete.

Capillary space—Void space in concrete resembling microscopic channels small enough to draw liquid water through them by the molecular attraction of the water adsorbed on their inner surfaces (capillarity).

Carbon black—A finely divided amorphous carbon used to color concrete; produced by burning natural gas in supply of air insufficient for complete combustion; characterized by a high oil absorption and a low specific gravity.

Carbonation—Reaction between carbon dioxide and a hydroxide or oxide to form a carbonate, especially in cement paste, mortar, or concrete; the reaction with calcium compounds to produce calcium carbonate.

Carriageway—In the United Kingdom, a term used in the same meaning as the word "road" in the United States.

Cast-in-place—Mortar or concrete which is deposited in the place where it is required to harden as part of the structure, as opposed to precast concrete.

Cast-in-place pile—A concrete pile concreted either with or without a casing in its permanent location, as distinguished from a precast pile.

Cast-in-situ—See **Cast-in-place.**

Cast stone—Concrete or mortar cast into blocks or small slabs in special molds so as to resemble natural building stone.

Castable refractory—A packaged, dry mixture of hydraulic cement, generally calcium-aluminate cement, and specially selected and proportioned refractory aggregates which, when mixed with water, will produce refractory concrete or mortar.

Catalyst—A substance that initiates a chemical reaction and enables it to proceed under milder conditions than otherwise required and which does not, itself, alter or enter into the reaction.

Catface—Blemish or rough depression in the finish plaster coat caused by variations in the base coat thickness.

Cathead—A notched wedge placed between two formwork members meeting at an oblique angle; a spindle on a hoist; the large, round retention nut used on she bolts.

Catwalk—A narrow elevated walkway.

Cavitation damage—Pitting of concrete caused by implosion, i.e., the collapse of vapor bubbles in flowing water which form in areas of low pressure and collapse as they enter areas of higher pressure.

Celite—A name used by Tornebohm (1897) to identify the calcium aluminoferrite constituent of portland cement. (See also **Alite, Belite, Felite**, and **Brownmillerite.**)

Cellular concrete—A lightweight product consisting of portland cement, cement-silica, cement-pozzolan, lime-pozzolan, or lime-silica pastes, or pastes containing blends of these ingredients and having a homogeneous void or cell structure, attained with gas-forming chemicals or foaming agents (for cellular concretes containing binder ingredients other than, or in addition to, portland cement, autoclave curing is usually employed).

Cellular construction—A method of constructing concrete elements in which part of the interior concrete is replaced by voids.

Cement, aluminous—See **Calcium-aluminate cement.**

Cement, bituminous—A black solid, semisolid, or liquid substance at natural air temperatures and appreciably soluble only in carbon disulfide or some volatile liquid hydrocarbon, being composed of mixed indeterminate hydrocarbons mined from natural deposits, produced as a residue in the distillation of petroleum, or obtained by the destructive distillation of coal or wood.

Cement, blended—A hydraulic cement consisting essentially of an intimate and uniform blend of granulated blast-furnace slag and hydrated lime; or an intimate and uniform blend of portland cement and granulated blast-furnace slag, portland cement and pozzolan, or portland blast-furnace slag cement and pozzolan, produced by intergrinding portland cement clinker with the other materials or by blending portland cement with the other materials, or a combination of intergrinding and blending.

Cement, expansive—See **Expansive cement.**

Cement, high-early-strength—Cement characterized by producing earlier strength in mortar or concrete than regular cement, referred to in United States as "Type III."

Cement, hydraulic—See **Hydraulic cement.**

Cement, low-alkali—A portland cement that contains a relatively small amount of sodium or potassium or both; in the United States a cement containing not more than 0.6 percent Na_2O equivalent, i.e., percent Na_2O + (0.658 x percent K_2O).

Cement, low-heat—A cement in which there is only limited generation of heat during setting, achieved by modifying the chemical composition of normal portland cement, referred to in United States as "Type IV."

Cement, masonry—A hydraulic cement for use in mortars for masonry construction, containing one or more of the following materials: portland cement, portland blast-furnace slag cement, portland-pozzolan cement, natural cement, slag cement or hydraulic lime; and in addition usually containing one or more materials such as hydrated lime, limestone, chalk, calcareous shell, talc, slag, or clay, as prepared for this purpose.

Cement, natural—A hydraulic cement produced by calcining a naturally occurring argillaceous limestone at a temperature below the sintering point and then grinding to a fine powder.

Cement, oil-well—Hydraulic cement suitable for use under high pressure and temperature in sealing water and gas pockets and setting casing during the drilling and repair of oil wells; often contains retarders to meet the requirements of use.

Cement, plastic—A special product manufactured for plaster and stucco application.

Cement, portland—A hydraulic cement produced by pulverizing clinker consisting essentially of hydraulic calcium silicates, and usually containing one or more of the forms of calcium sulfate as an interground addition.

Cement, portland blast-furnace slag—A hydraulic cement consisting essentially of an intimately interground mixture of portland cement clinker and granulated blast-furnace slag or an intimate and uniform blend of portland cement and fine granulated blast-furnace slag in which the amount of the slag constituent is within specified limits.

Cement, portland-pozzolan—A hydraulic cement consisting essentially of an intimate and uniform blend of portland cement or portland blast-furnace slag cement and fine pozzolan produced by intergrinding portland-cement clinker and pozzolan, by blending portland cement or portland blast-furnace slag cement and finely divided pozzolan, or a combination of intergrinding and blending, in which the pozzolan constituent is within specified limits.

Cement, self-stressing—See **Expansive cement.**

Cement, shrinkage-compensating—See **Expansive cement.**

Cement, slag—A hydraulic cement consisting essentially of an intimate and uniform blend of granulated blast-furnace slag and hydrated lime in which the slag constituent is more than a specified minimum percentage.

Cement, sulfate-resistant—Portland cement, low in tricalcium aluminate, to reduce susceptibility of concrete to attack by dissolved sulfates in water or soils, designated "Type V" in United States.

Cement, sulfoaluminate—See **Expansive cement.**

Cement, supersulfated—A hydraulic cement made by intimately intergrinding a mixture of granulated blast-furnace slag, calcium sulfate, and a small amount of lime, cement, or cement clinker; so named because the equivalent content of sulfate exceeds that for portland blast-furnace slag cement.

Cement, white—Portland cement which hydrates to a white paste; made from raw materials of low iron content the clinker for which is fired by a reducing flame.

Cement-aggregate ratio—The ratio of cement to total aggregate either by weight or volume.

Cement bacillus—See **Ettringite.**

Cement content—Quantity of cement contained in a unit volume of concrete or mortar, preferably expressed as weight.

Cement factor—See **Cement content.**

Cement gel—The colloidal material that makes up the major portion of the porous mass of which mature hydrated cement paste is composed.

Cement gun—A machine for pneumatic placement of mortar or small aggregate concrete; in the "Dry Gun" water from a separate hose meets the dry material at the nozzle of the gun; with the "Wet Gun" the delivery hose conveys the premixed mortar or concrete. (See also **Shotcrete.**)

Cement kiln—See **Kiln, cement.**

Cement paint—A paint consisting generally of white portland cement and water, pigments, hydrated lime, water repellents, or hygroscopic salts.

Cement plaste, neat—See **Neat cement paste.**

Cement plaster—See **Stucco** and **Plaster.**

Cement rock—Natural impure limestone which contains the ingredients for production of portland cement in approximately the required proportions.

Cementation process—The process of injecting cement grout under pressure into certain types of ground (e.g., gravel, fractured rock) to solidify it.

Cementitious—Having cementing properties.

Center-matched—Tongue-and-groove lumber with the tongue and groove at the center of the piece rather than offset as in standard matched. (See also **Standard matched.**)

Centering—Specialized falsework used in the construction of arches, shells, and space structures, or any continuous structure where the entire falsework is lowered (struck or decentered) as a unit to avoid the introduction of injurious stress in any part of the structure. (See also **Falsework.**)

Central-mixed concrete—Concrete which is completely mixed in a stationary mixer from which it is transported to the delivery point.

Central mixer—A stationary concrete mixer from which the freshly mixed concrete is transported to the work.

Centrifugally cast concrete—See **Spun concrete.**

Ceramic bond—The development of fired strength as a result of thermo-chemical reactions between materials exposed to temperatures approaching the fusion point of the mixture such as that which may occur, under these conditions, between calcium-aluminate cement and a refractory aggregate.

Chair—See **Bar support**.

Chalk—A soft limestone composed chiefly of the calcareous remains of marine organisms.

Chalking—Disintegration of coatings such as a cement paint, manifested by the presence of a loose powder evolved from the paint at, or just beneath, the surface.

Charging—Introducing, feeding, or loading materials into a concrete or mortar mixer, furnace, or other container or receptacle where they will be further treated or processed.

Checking—Development of shallow cracks at closely spaced but irregular intervals on the surface of mortar or concrete.

Chemical bond—Bond between materials that is the result of cohesion and adhesion developed by chemical reaction.

Chemically prestressed concrete—Concrete made with expansive cement and reinforcing under conditions such that the expansion of the cement induces tensile stress in the reinforcing so as to produce prestressed concrete.

Chemically prestressing cement—A type of expansive cement containing a higher percentage of expansive component than shrinkage compensating cement which, when used in concretes with adequate internal or external restraint, will expand sufficiently, due to chemical reactions within the matrix, to develop the stresses necessary for prestressing the concrete.

Chert—A very fine grained siliceous rock characterized by hardness and conchoidal fracture in dense varieties, the fracture becoming splintery and the hardness decreasing in porous varieties, and in a variety of colors; it is composed of silica in the form of chalcedony, cryptocrystalline or microcrystalline quartz, or opal, or combinations of any of these.

Chipping—Treatment of a hardened concrete surface by chiseling.

Chips—Broken fragments of marble or other mineral aggregate screened to specified sizes.

Chordus moldus—See **Modulus of elasticity**.

Chute—A sloping trough or tube for conducting concrete, cement, aggregate, or other free flowing materials from a higher to a lower point.

Clamp—See **Tie Coupler**.

Class (of concrete)—An arbitrary characterization of concrete of various qualities or usages, usually by compressive strength.

Clay—Natural mineral material having plastic properties and composed of very fine particles; the clay mineral fraction of a soil is usually considered to be the portion consisting of particles finer than 2 μm; clay minerals are essentially hydrous aluminum silicates or occasionally hydrous magnesium silicates.

Clay content—Percentage of clay by dry weight of a heterogeneous material, such as a soil or a natural concrete aggregate.

Cleanout—An opening in the forms for removal of refuse, to be closed before the concrete is placed; a port in tanks, bins, or other receptacles for inspection and cleaning.

Cleanup—Treatment of horizontal construction joints to remove all surface material and contamination down to a condition of cleanness corresponding to that of a freshly broken surface of concrete.

Cleat—Small board used to connect formwork members or used as a brace. (See also **Batten**.)

Climbing form—A form which is raised vertically for succeeding lifts of concrete in a given structure.

Clinker—A partially fused product of a kiln, which is ground to make cement; also other vitrified or burnt material.

Clip—Wire or sheet-metal device used to attach various types of lath to supports or to secure adjacent lath sheets.

Closed-circuit grouting—Injection of grout into a hole intersecting fissures or voids which are to be filled at such volume and pressure that grout input to the hole is greater than the grout take of the surrounding formation, excess grout being returned to the pumping plant for recirculation.

Coarse aggregate—Aggregate predominantly retained on the U.S. Standard No. 4 (4.75 mm) sieve; or that portion of an aggregate retained on the No. 4 (4.75 mm) sieve. (See also **Aggregate**.)

Coarse aggregate factor—The ratio, expressed as a decimal, of the amount (weight or solid volume) of coarse aggregate in a unit volume of well-proportioned concrete to the amount of dry-rodded coarse aggregate compacted into the same volume (b/b_0).

Coarse-grained soil—Soil in which the larger grain sizes, such as sand and gravel predominate.

Coat—A film or layer as of paint or plaster applied in a single operation.

Coating—Material applied to a surface by brushing, dipping, mopping, spraying, trowelling, etc., to preserve, protect, decorate, seal, or smooth the substrate; also refers to foreign or deleterious substances found adhering to aggregate particles.

Cobble—In geology, a rock fragment between 2½ and 10 in. (64 and 256 mm) in diameter; as applied to coarse aggregate for concrete, the material in the nominal size range 3 to 6 in. (75 to 150 mm).

Cobblestone—A rock fragment, usually rounded or semirounded, with an average dimension between 3 and 12 in. (75 and 300 mm).

Coefficient of subgrade friction—The coefficient of friction between a grade slab and its subgrade; used to estimate shrinkage reinforcing steel requirements by calculating stresses induced in the concrete by its shrinkage and the subgrade restraint.

Coefficient of subgrade reaction—See **Modulus of subgrade reaction**.

Coefficient of thermal expansion—Change in linear dimension per unit length or change in volume per unit volume per degree of temperature change.

Coefficient of variation (V)—The standard deviation expressed as a percentage of the average. (See also **Standard deviation**.)

Cold face—The surface of a refractory section not exposed to the source of heat.

Cold joint—A joint or discontinuity resulting from a delay in placement of sufficient time to preclude a union of the material in two successive lifts.

Cold joint lines—Visible lines on the surfaces of formed concrete indicating the presence of joints where one layer of concrete had hardened before subsequent concrete was placed. (See also **Cold joint**.)

Cold strength—The compressive or flexural strength of refractory concrete determined prior to drying or firing.

Cold-water paint—A paint in which the binder or vehicle portion is composed of latex, casein, glue, or some similar material dissolved or dispersed in water.

Col

Colemanite—A mineral, hydrated calcium borate ($Ca_2B_6O_{11} \cdot 5H_2O$). (See also **Boron-loaded concrete.**)

Colloid—A substance that is in a state of division preventing passage through a semipermeable membrane, consisting of particles ranging from 10^{-5} to 10^{-7} cm in diameter.

Colloidal concrete—Concrete of which the aggregate is bound by colloidal grout.

Colloidal grout—A grout which has artificially induced cohesiveness or ability to retain the dispersed solid particles in suspension.

Colloidal particle—An electrically charged particle, generally smaller than 200 μm, dispersed in a second continuous medium.

Colorimetric value—An indication of the amount of organic impurities present in fine aggregate.

Column—A member used primarily to support axial compression loads and with a height of at least three times its least lateral dimension.

Column, long—A column whose load capacity is limited by buckling rather than strength. (See also **Column, slender.**)

Column, short—A column whose load capacity is limited by strength rather than buckling; a column which is customarily so stocky and sufficiently restrained that at least 95 percent of the cross-sectional strength can be developed.

Column, slender—A column whose load capacity is reduced by the increased eccentricity caused by secondary deflection moments.

Column capital—An enlargement of the end of a column designed and built to act as an integral unit with the column and flat slab and increase the shearing resistance.

Column clamp—Any of various types of tying or fastening units to hold column form sides together.

Column side—One of the vertical panel components of a column form.

Column strip—The portion of a flat slab over the columns and consisting of the two adjacent quarter panels on each side of the column center line.

Combined aggregate grading—Particle size distribution of a mixture of fine and coarse aggregate.

Combined footing—A structural unit or assembly of units supporting more than one column.

Come-along—

1. A hoe-like tool with a blade about 4 in. (10 cm) high and 20 in. (50 cm) wide and curved from top to bottom, used for spreading concrete.
2. A colloquial name for a device (load binder) used to tighten chains holding loads in place on a truck bed.

Compacting factor—The ratio obtained by dividing the observed weight of concrete which fills a container of standard size and shape when allowed to fall into it under standard conditions of test, by the weight of fully compacted concrete which fills the same container.

Compaction—The process whereby the volume of freshly placed mortar or concrete is reduced to the minimum practical space usually by vibration, centrifugation, tamping, or some combination of these; to mold it within forms or molds and around embedded parts and reinforcement, and to eliminate voids other than entrained air. (See also **Consolidation.**)

Composite column—A concrete compression member reinforced longitudinally with structural steel shapes, pipe or tubing with or without longitudinal reinforcing bars.

Composite concrete flexural members—Concrete flexural members consisting of concrete elements constructed in separate placements but so interconnected that the elements respond to loads as a unit.

Composite construction—A type of construction using members produced by combining different materials (e.g., concrete and structural steel), or members produced by combining cast-in-place and precast concrete such that the combined components act together as a single member.

Composite pile—A pile made up of different materials, usually concrete and wood, or steel fastened together end to end, to form a single pile.

Composite sample—Sample obtained by blending two or more individual samples of a material.

Compound, curing—See **Curing compound, Membrane curing.**

Compound, joint sealing—An impervious material used to fill joints in pavements or structures.

Compound, sealing—An impervious material applied as a coating or to fill joints or cracks in concrete or mortar. (See also **Joint sealant.**)

Compound, "waterproofing"— Material used to impart water repellency to a structure or a constructional unit.

Compression flange—The widened portion of an I, T, or similar cross section beam which is shortened or compressed by bending under normal loads, such as the horizontal portion of the cross section of a simple span T-beam.

Compression member—Any member in which the primary stress is longitudinal compression.

Compression reinforcement—Reinforcement designed to carry compressive stresses. (See also **Stress.**)

Compression test—Test made on a test specimen of mortar or concrete to determine the compressive strength; in the United States, unless otherwise specified, compression tests of mortars are made on 2-in. (50-mm) cubes and compression tests of concrete are made on cylinders 6 in. (152 mm) in diameter and 12 in. (305 mm) high.

Compressive strength—The measured maximum resistance of a concrete or mortar specimen to axial loading; expressed as force per unit cross-sectional area; or the specified resistance used in design calculations, in the U.S. customary units of measure expressed in pounds per square inch (psi) and designated f_c'.

Compressive strength, average f_c'—The term used to describe the average compressive strength of a given class or strength level of concrete; in ACI 214, defined as average compressive strength required to statistically meet a designated specific strength, f_c'.

Concentric tendons—Tendons following a line coincident with the gravity axis of the prestressed concrete member.

Concordant tendons—Tendons in statically indeterminate structures which are coincident with the pressure line produced by the tendons; such tendons do not produce secondary moments.

Concrete—A composite material which consists essentially of a binding medium within which are embedded particles or fragments of aggregate; in portland cement concrete, the binder is a mixture of portland cement and water.

Concrete, aerated—See **Concrete, foamed.**

Concrete, asphalt—A mixture of asphalt and aggregate.
Concrete, colloidal—See **Colloidal concrete**.
Concrete, cyclopean—See **Cyclopean concrete**.
Concrete, dense—Concrete containing a minimum of voids.
Concrete, dry-packed—See **Dry-packed concrete**.
Concrete, exposed—See **Exposed concrete**.
Concrete, fair face—See **Fair face concrete**.
Concrete, fat—See **Fat concrete**.
Concrete, fibrous—Concrete containing, dispersed, randomly oriented fibers.
Concrete, field—Concrete delivered or mixed, placed, and cured on the job site.
Concrete, foamed—Concrete made very light and cellular by the addition of a prepared foam or by generation of gas within the unhardened mixture.
Concrete, gap-graded—See **Gap-graded concrete**.
Concrete, granolithic—See **Granolithic concrete**.
Concrete, green—Concrete which has set but not appreciably hardened.
Concrete, heavy—See **High-density concrete**.
Concrete, high-density—See **High-density concrete**.
Concrete, in-situ (also **cast-in-place**)—Concrete which is deposited in the place where it is required to harden as part of the structure, as opposed to precast concrete.
Concrete, lean—See **Lean concrete**.
Concrete, lightweight—See **Lightweight concrete**.
Concrete, low-density—See **Low-density concrete**.
Concrete, mass—See **Mass concrete**.
Concrete, monolithic—See **Monolithic concrete**.
Concrete, no-fines—See **No-slump concrete**.
Concrete, normal-weight—Concrete having a unit weight of approximately 150 lb per cu ft (2400 kg per cu m) made with aggregates of normal weight.
Concrete pile—See **Cast-in-place pile** and **Precast pile**.
Concrete, precast—See **Precast concrete**.
Concrete, prepacked—See **Concrete, preplaced-aggregate.**
Concrete, preplaced-aggregate—Concrete produced by placing coarse aggregate in a form and later injecting a portland cement-sand grout, usually with admixtures, to fill the voids.
Concrete, prestressed—See **Prestressed concrete**.
Concrete, pumped—See **Pumped concrete**.
Concrete, ready-mixed—See **Ready-mixed concrete.**
Concrete, refractory—See **Refractory concrete**.
Concrete, reinforced—See **Reinforced concrete**.
Concrete, spun—See **Spun concrete**.
Concrete, structural—Concrete used to carry structural load or to form an integral part of a structure; concrete of a quality specified for structural use; concrete used solely for protective cover, fill, or insulation is not considered structural concrete.
Concrete, structural lightweight—Structural concrete made with lightweight aggregate; the unit weight usually is in the range of 90 to 115 lb per cu ft (1440 to 1850 kg per cu m).
Concrete, terrazzo—Marble-aggregate concrete that is cast-in-place or precast and ground smooth for decorative surfacing purposes on floors and walls.
Concrete, transit-mixed—See **Transit-mixed concrete**.
Concrete, translucent—A combination of glass and concrete used together in precast or prestressed panels.
Concrete, vacuum—See **Vacuum concrete**.
Concrete, vibrated—See **Vibrated concrete**.
Concrete, visual—See **Exposed concrete**.

Concrete breaker—A compressed-air tool specially designed and constructed to break up concrete.
Concrete cart—See **Buggy**.
Concrete containment structure—A composite concrete and steel assembly that is designed as an integral part of a pressure retaining barrier which in an emergency prevents the release of radioactive or hazardous effluents from nuclear power plant equipment enclosed therein.
Concrete finishing machine—A machine mounted on flanged wheels which rides on the forms or on specially set tracks, used to finish surfaces such as those of pavements; or a portable power driven machine for floating and finishing of floors and other slabs.
Concrete paver—A concrete mixer, usually mounted on crawler tracks, which mixes and places concrete pavement on the subgrade.
Concrete pile—A precast reinforced or prestressed concrete pile driven into the ground by a pile driver or otherwise placed. (See also **Cast-in-place pile**.)
Concrete pump—An apparatus which forces concrete to the placing position through a pipeline or hose.
Concrete reactor vessel—A composite concrete and steel assembly that functions as a component of the principal pressure-containing barrier for the nuclear fuel's primary heat extraction fluid (primary coolant).
Concrete spreader—A machine, usually carried on side forms or on rails parallel thereto, designed to spread concrete from heaps already dumped in front of it, or to receive and spread concrete in a uniform layer.
Concrete strength—See **Compressive strength, Fatigue strength, Flexural strength, Shear strength, Splitting tensile strength, Tensile strength, Ultimate strength,** and **Yield strength**.
Concrete vibrating machine—A machine which compacts a layer of freshly mixed concrete by vibration.
Conductivity, thermal—See **Thermal conductivity**.
Cone, slump—See **Slump cone** and **Slump**.
Cone bolt—A form of tie rod for wall forms with cones at each end inside the forms so that a bolt can act as a spreader as well as a tie.
Confined concrete—Concrete containing closely spaced special transverse reinforcement which is provided to restrain the concrete in directions perpendicular to the applied stresses.
Confined region—Region with transverse reinforcement within beam-column joints.
Consistency—The relative mobility or ability of freshly mixed concrete or mortar to flow; the usual measurements are slump for concrete, flow for mortar or grout, and penetration resistance for neat cement paste.
Consistency factor—A measure of grout fluidity roughly analogous to viscosity, which describes the ease with which grout may be pumped into pores or fissures; usually a laboratory measurement in which consistency is reported in degrees of rotation of a torque viscosimeter in a specimen of grout.
Consistometer—An apparatus for measuring the consistency of cement pastes, mortars, grouts, or concretes.
Consolidation—The process of inducing a closer arrangement of the solid particles in freshly mixed concrete or mortar during placement by the reduction of voids; usually by vibration, centrifugation, tamping, or some combination of these actions; also applicable to similar manipulation of other cementitious mixtures, soils, aggregates, or the like. (See also **Compaction**.)

Construction joint—The surface where two successive placements of concrete meet, across which it is desirable to develop and maintain bond between the two concrete placements, and through which any reinforcement which may be present is not interrupted. (See also **Contraction joint.**)

Construction loads—The loads to which a permanent or temporary structure is subjected during construction.

Contact ceiling—A ceiling which is secured in direct contact with the construction above without use of furring.

Contact pressure—Pressure acting at and perpendicular to the contact area between footing and soil, produced by the weight of the footing and all forces acting on it.

Contact splice—A means of connecting reinforcing bars in which the bars are lapped and in direct contact. (See also **Lap splice.**)

Containment grouting—See **Perimeter grouting**.

Continuous beam—See **Continuous slab or beam**.

Continuous footing—A combined footing of prismatic or truncated shape, supporting two or more columns in a row.

Continuous grading—A particle size distribution in which all intermediate size fractions are present, as opposed to gap-grading.

Continuous mixer—A mixer into which the ingredients of the mixture are fed without stopping, and from which the mixed product is discharged in a continuous stream.

Continuous slab or beam—A slab or beam which extends as a unit over three or more supports in a given direction.

Continuously reinforced pavement—A pavement with continuous longitudinal steel reinforcement and no intermediate transverse expansion or contraction joints.

Contraction (or **Expansion**), **of concrete**—The sum of volume changes occurring as the result of all processes affecting the bulk volume of a mass of concrete. (See also **Shrinkage.**)

Contraction joint—Formed, sawed, or tooled groove in a concrete structure to create a weakened plane and regulate the location of cracking resulting from the dimensional change of different parts of the structure. (See also **Isolation joint.**)

Contraction-joint grouting—Injection of grout into contraction joints.

Control factor—The ratio of the minimum compressive strength to the average compressive strength.

Control joint—See **Contraction joint**.

Control-joint grouting—See **Contraction-joint grouting**.

Conventional design—Design procedure using moments or stresses determined by widely accepted methods.

Conveying hose—See **Delivery hose**.

Conveyor—A device for moving materials; usually a continuous belt, an articulated system of buckets, a confined screw, or a pipe through which material is moved by air or water.

Coping—The material or units used to form a cap or finish on top of a wall, pier, pilaster, or chimney.

Coquina—A type of limestone formed of sea shells in loose or weakly cemented condition, found along present or former shorelines; used as a calcareous raw material in cement manufacture and other industrial operations.

Corbel—A projection from the face of a beam, girder, column, or wall used as a beam seat or a decoration.

Core—

1. The soil material enclosed within a tubular pile after driving (it may be replaced with concrete).
2. The mandrel used for driving casings for cast-in-place piles.
3. A structural shape used to internally reinforce a drilled-in-caisson.
4. A cylindrical sample of hardened concrete or rock obtained by means of a core drill.
5. The molded open space in a concrete masonry unit. See also **Blockout**.

Core test—Compression test on a concrete sample cut from hardened concrete by means of a core drill.

Cored beam—A beam whose cross section is partially hollow, or a beam from which cored samples of concrete have been taken.

Coring—The act of obtaining cores from concrete structures or rock foundations.

Corner reinforcement—Metal reinforcement for plaster at re-entrant corners to provide continuity between two intersecting planes; or concrete reinforcement used at wall intersections or near corners of square or rectangular openings in walls, slabs, or beams.

Corrosion—Disintegration or deterioration of concrete or reinforcement by electrolysis or by chemical attack.

Cotton mats—Cotton-filled quilts fabricated for use as a water-retaining covering in curing concrete surfaces.

Coupler—

1. A device for connecting reinforcing bars or prestressing tendons end to end.
2. A device for locking together the component parts of a tubular metal scaffold (also known as a **Clamp**).

Coupling pin—An insert device used to connect lifts or tiers or formwork scaffolding vertically.

Course—In concrete construction, a horizontal layer of concrete, usually one of several making up a lift; in masonry construction, a horizontal layer of block or brick. (See also **Lift.**)

Cover—In reinforced concrete, the least distance between the surface of the reinforcement and the outer surface of the concrete.

Cover block—See **Spacer**.

Crack-control reinforcement—Reinforcement in concrete construction designed to prevent opening of cracks, often effective in limiting them to uniformly distributed small cracks.

Cracked section—A section designed or analyzed on the assumption that concrete has no resistance to tensile stress.

Cracking load—The load which causes tensile stress in a member to exceed the tensile strength of the concrete.

Craze cracks—Fine, random cracks or fissures caused by shrinkage which may appear in a surface of plaster, cement paste, mortar, or concrete.

Crazing—The development of craze cracks; the pattern of craze cracks existing in a surface. (See also **Checking.**)

Creep—Time-dependent deformation due to sustained load.

Crimped wire—Wire deformed into a curve which approximates a sine curve as a means of increasing the capacity of the wire to bond to concrete; also welded wire fabric crimped to provide an integral chair. (See also **Deformed reinforcement.**)

Cross bracing—A system of members which connect frames or panels of scaffolding laterally to make a tower or continuous structure.

Cross joint—The joint at the end of individual formboards between sub-purlins.

Cross section—The section of a body perpendicular to a given axis of the body; a drawing showing such a section.

Cross-tee—A light-gage metal member resembling an upside-down "tee" used to support the abutting ends of form-boards in insulating concrete roof constructions

Crush plate—An expendable strip of wood attached to the edge of a form or intersection of fitted forms, to protect the form from damage during prying, pulling, or other stripping operations. (See also **Wrecking strip**.)

Crushed gravel—The product resulting from the artificial crushing of gravel with a specified minimum percentage of fragments having one or more faces resulting from fracture. (See also **Coarse aggregate**.)

Crushed stone—The product resulting from the artificial crushing of rocks, boulders, or large cobblestones, substantially all faces of which possess well-defined edges and have resulted from the crushing operation. (See also **Coarse aggregate**.)

Crusher-run aggregate—Aggregate that has been broken in a mechanical crusher and has not been subjected to any subsequent screening process.

C/S ratio—The molar or weight ratio, whichever is specified, of calcium oxide to silicon dioxide; usually of binder materials cured in an autoclave.

Cube strength—The load per unit area at which a standard cube fails when tested in a specified manner.

Cubical piece (of aggregate)—One in which length, breadth and thickness are approximately equal.

Cumulative batching—Measuring more than one ingredient of a batch in the same container by bringing the batcher scale into balance at successive total weights as each ingredient is accumulated in the container.

Curb form—A retainer or mold used in conjunction with a curb tool to give the necessary shape and finish to a concrete curb.

Curb tool—A tool used to give the desired finish and shape to the exposed surfaces of a concrete curb.

Curing—Maintenance of humidity and temperature of freshly placed concrete during some definite period following placing, casting, or finishing to assure satisfactory hydration of the cementitious materials and proper hardening of the concrete.

Curing, electrical—A system in which a favorable temperature is maintained in freshly-placed concrete by supplying heat generated by electrical resistance.

Curing, steam—See **Steam curing**.

Curing agent—Catalyst, hardener. (See also **Catalyst, Hardener**.)

Curing blanket—A built-up covering of sacks, matting, hessian, straw, waterproof paper, or other suitable material placed over freshly finished concrete. (See also **Burlap**.)

Curing compound—A liquid that can be applied as a coating to the surface of newly placed concrete to retard the loss of water or, in the case of pigmented compounds, also to reflect heat so as to provide an opportunity for the concrete to develop its properties in a favorable temperature and moisture environment. (See also **Curing**.)

Curing cycle—See **Autoclave cycle** and **Steam-curing cycle**.

Curing delay—See **Presteaming period**.

Curing kiln—See **Steam curing room**.

Curing membrane—See **Membrane curing** and **Curing compound**.

Def

Curling—The distortion of an originally essentially linear or planar member into a curved shape such as the warping of a slab due to creep or to differences in temperature or moisture content in the zones adjacent to its opposite faces.

Curtain—A vertically-placed mat of vertical and horizontal reinforcing steel in a member such as a wall; known as a double curtain (of reinforcement) when a mat is at each face.

Curtain grouting—Injection of grout into a subsurface formation in such a way as to create a zone of grouted material transverse to the direction of anticipated water flow.

Curvature friction—Friction resulting from bends or curves in the specified prestressing cable profile.

Cutting screed—Sharp edged tool used to trim shotcrete to finished outline. (See also **Rod**.)

Cyclopean concrete—Mass concrete in which large stones, each of 100 lb (50 kg) or more, are placed and embedded in the concrete as it is deposited. (See also **Rubble concrete**.)

Cylinder strength—See **Compressive strength**.

D

Dampproofing—Treatment of concrete or mortar to retard the passage or absorption of water, or water vapor, either by application of a suitable coating to exposed surfaces, or by use of a suitable admixture or treated cement, or by use of pre-formed films such as polyethylene sheets under slabs on grade. (See also **Vapor barrier**.)

Darby—A hand-manipulated straightedge, usually 3 to 8 ft (1 to 2.5 m) long, used in the early stage leveling operations of concrete or plaster, preceding supplemental floating and finishing.

Dash-bond coat—A thick slurry of portland cement, sand, and water flicked on surfaces with a paddle or brush to provide a base for subsequent portland cement plaster coats; sometimes used as a final finish on plaster.

D-cracking—The progressive formation on a concrete surface of a series of fine cracks at rather close intervals, often of random patterns, but in slabs on grade paralleling edges, joints, and cracks and usually curving across slab corners. (Also termed **D-cracks** and **D-line cracks**.)

Dead end—In the stressing of a tendon from one end only, the end opposite that to which stress is applied.

Dead-end anchorage—The anchorage at that end of a tendon which is opposite the jacking end.

Dead load—A constant load that in structures is due to the mass of the members, the supported structure, and permanent attachments or accessories.

Deadman—An anchor for a guy line, usually a beam, block, or other heavy item buried in the ground, to which a line is attached.

Decenter—To lower or remove centering or shoring.

Deck—The form on which concrete for a slab is placed, also the floor or roof slab itself. (See also **Bridge deck**.)

Decking—Sheathing material for a deck or slab form.

Deflected tendons—Tendons which have a trajectory that is curved or bent with respect to the gravity axis of the concrete member.

Deflection—A variation in position or shape of a structure or structural element due to effects of loads or volume

Def

change, usually measured as a linear deviation from an established plane rather than an angular variation.

Deformation—A change in dimension or shape due to stress. (See also **Time dependent deformation.**)

Deformed bar—A reinforcing bar with a manufactured pattern of surface ridges which provide a locking anchorage with surrounding concrete.

Deformed plate—A flat piece of metal, thicker than ¼ in. (6 mm), having horizontal deformations or corrugations; used in construction to form a vertical joint and provide a mechanical interlock between adjacent sections.

Deformed reinforcement—Metal bars, wire, or fabric with a manufactured pattern of surface ridges which provide a locking anchorage with surrounding concrete.

Deformed tie bar—See **Tie bar.**

Dehydration—Removal of chemically bound, adsorbed, or absorbed water from a material.

Deicer—A chemical, such as sodium or calcium chloride, used to melt ice or snow on slabs and pavements, such melting being due to depression of the freezing point.

Delamination—A separation along a plane parallel to a surface as in the separation of a coating from a substrate or the layers of a coating from each other, or in the case of a concrete slab, a horizontal splitting, cracking, or separation of a slab in a plane roughly parallel to, and generally near,the upper surface; found most frequently in bridge decks and caused by the corrosion of reinforcing steel or freezing and thawing; similar to spalling, scaling or peeling except that delamination affects large areas and can often only be detected by tapping.

Delay—See **Prestreaming period**.

Delivery hose—Hose through which shotcrete, grout, or pumped concrete or mortar passes; also known as material hose or conveying hose.

Demolding—Removal of molds from concrete test specimens or precast products. (See also **Strip.**)

Dense concrete—See **Concrete, dense.**

Dense-graded aggregate—Aggregates graded to produce low void content and maximum weight when compacted.

Density—Weight per unit volume. (See also **Specific gravity.**)

Density (dry)—The weight per unit volume of a dry substance at a stated temperature. (See also **Specific gravity.**)

Density control—Control of density of concrete in field construction to insure that specified values as determined by standard tests are obtained.

Design load—Load, multiplied by appropriate load factor, used to proportion members.

Design strength—The load-bearing capacity of a member computed on the basis of strain compatibility between the concrete and the reinforcement, the resulting design stresses being taken as the strength of concrete and the yield stress of steel respectively to compute the ultimate strength of a section.

Deterioration—Disintegration or chemical decomposition of a material during test or service exposure. (See also **Disintegration.**)

Detritus—Loose material produced by the disintegration of rocks through geological agencies or processes simulating those of nature.

Development bond stress—See **Anchorage bond stress.**

Development length—The length of embedded reinforcement required to develop the design strength of the reinforcement at a critical section; formerly called bond length.

Devil's float—A wooden float with two nails protruding from the toe, used to roughen the surface of a brown plaster coat. (See also **Texturing.**)

Diagonal crack—An inclined crack caused by shear stress, usually at about 45 degrees to the neutral axis of a concrete member; or a crack in a slab, not parallel to lateral or longitudinal dimension.

Diagonal cracking—Development of diagonal cracks. (See also **Diagonal tension.**)

Diagonal tension—The principal tensile stress resulting from the combination of normal and shear stresses acting upon a structural element.

Diametral compression test—See **Splitting tensile test.**

Diamond mesh—A metallic fabric having rhomboidal openings in a geometric pattern. (See also **Expanded metal lath.**)

Diatomaceous earth—A friable earthy material composed of nearly pure hydrous amorphous silica (opal) and consisting essentially of the frustules of the microscopic plants called diatoms.

Dicalcium silicate—A compound having the composition $2CaO \cdot SiO_2$, abbreviated C_2S, that occurs in portland-cement clinker. (See also **Belite.**)

Differential thermal analysis (DTA)—Indication of thermal reaction by differential thermocouple recording of temperature changes in a sample under investigation compared with those of a thermally passive control sample, that is heated uniformly and simultaneously.

Diffusivity, thermal—See **Thermal diffusivity**.

Dilation—An expansion of concrete during cooling or freezing generally calculated as the maximum deviation from the normal thermal contraction predicted from the length change-temperature curve or length change-time curve established at temperatures before initial freezing.

Diluent—A substance, liquid or solid, mixed with the active constituents of a formulation to increase the bulk or lower the concentration.

Direct dumping—Discharge of concrete directly into place from crane bucket or mixer.

Discoloration—Departure of color from that which is normal or desired.

Disintegration—Deterioration into small fragments or particles due to any cause.

Dispersant—A material which deflocculates or disperses finely ground materials by satisfying the surface energy requirements of the particles; used as a slurry thinner or grinding aid.

Dispersing agent—An agent capable of increasing the fludity of pastes, mortars, or concretes by reduction of inter-particle attraction.

Divider strips—In terrazzo work, nonferrous metal or plastic strips of different thicknesses, and embedded depths usually ⅝ to 1¼ in. (10 to 40 mm), used to form panels in the topping.

D-line cracks—See **D-cracking**.

Dolomite—A mineral having a specific crystal structure and consisting of calcium carbonate and magnesium carbonate in equivalent chemical amounts which are 54.27 and 45.73 percent by weight, respectively; a rock containing dolomite as the principal constituent.

Dome—Square prefabricated pan form used in two-way (waffle) concrete joist floor construction.

Double headed nail—A nail with two heads at, or near, one end to permit easy removal; widely used in concrete formwork.

Double T-beam—A precast concrete member composed of two beams and a top slab projecting on both sides; also a flat slab panel with projecting stems.

Double-up—A method of plastering characterized by application in successive operations with no setting or drying time between coats.

Doughnut (Donut)—A large washer of any shape to increase bearing area of bolts and ties; also a round concrete spacer with hole in the center to hold bars the desired distance from the forms.

Dowel—A steel pin, commonly a plain round steel bar, which extends into two adjoining portions of a concrete construction, as at a joint in a pavement slab, so as to connect the portions and transfer shear loads. Also, as used in the construction of column and wall sections, a deformed steel reinforcing bar placed so as to transmit tension or compression as well as shear loads.

Dowel deflection—Deflection caused by the transverse load imposed on a dowel.

Dowel lubricant—Lubricating material applied to bars in expansion joints to reduce bond with the concrete and promote unrestrained longitudinal movement.

Dowel shear—The force applied in the plane of the cross section of the dowel.

Drainage—The interception and removal of water from on or under an area or roadway; the process of removing surplus ground or surface water artificially; a general term for gravity flow of liquids in conduits.

Drainage fill—Base course of granular material placed between floor slab and sub-grade to impede capillary rise of moisture. Also, lightweight concrete placed on floors or roofs to promote drainage.

Draped tendons—See **Deflected tendons**.

Dried strength—The compressive or flexural strength of refractory concrete determined within 3 hrs after first drying in an oven at 220 to 230 F (105 to 110 C) for a specified time.

Drier—Chemical which promotes oxidation or drying of a paint or adhesive.

Drip—A transverse groove in the underside of a projecting piece of wood, stone, or concrete to prevent water from flowing back to a wall.

Dropchute—A device used to confine or to direct the flow of a falling stream of fresh concrete.

1. **Dropchute, articulated**—A device consisting of a succession of tapered metal cylinders so designed that the lower end of each cylinder fits into the upper end of the one below.

2. **Dropchute, flexible**—A device consisting of a heavy, rubberized canvas, or plastic, collapsible tube.

Drop-in beam—A simple beam, usually supported by cantilever arms, with joints so arranged that it is installed by lowering into position.

Drop panel—The thickened structural portion of a flat slab in the area surrounding column, column capital, or bracket, in order to reduce the intensity of stresses.

Drop panel form—A retainer or mold so erected as to give the necessary shape, support, and finish to a drop panel.

Dry-batch weight—The weight of the materials, excluding water, used to make a batch of concrete.

Dry mix—A concrete, mortar, or plaster mixture, commonly sold in bags, containing all components except water; also a concrete of near zero slump or less.

Dry-mix shotcrete—Pneumatically conveyed shotcrete in which most of the mixing water is added at the nozzle. (See also **Pneumatic feed**.)

Dry mixing—Blending of the solid materials for mortar or concrete prior to adding the mixing water.

Dry pack—Concrete or mortar mixtures deposited and consolidated by dry packing.

Dry packed concrete—Concrete placed by dry packing.

Dry packing—Placing of zero slump, or near zero slump, concrete, mortar, or grout by ramming into a confined space.

Dry process—In the manufacture of cement, the process in which the raw materials are ground, conveyed, blended, and stored in a dry condition. (See also **Wet process**.)

Dry-rodded volume—The bulk volume occupied by a dry aggregate compacted by rodding under the standardized conditions used in measuring unit weight of aggregate.

Dry-rodded weight—Weight per unit volume of dry aggregate compacted by rodding under standardized conditions; used in measuring unit weight of aggregate.

Dry rodding—In measurement of the weight per unit volume of coarse aggregates, the process of compacting dry material in a calibrated container by rodding under standardized conditions.

Dry-shake—A dry mixture of cement and fine aggregate, which is distributed evenly on an unformed surface after water has largely disappeared following the strike-off, and then worked in by floating.

Dry-tamp process—See **Dry packing**.

Dry topping—See **Dry-shake**.

Dry-volume measurement—Measurement of the ingredients of grout, mortar, or concrete by their bulk volume.

Drying shrinkage—Contraction caused by moisture loss. (See also **Shrinkage**.)

Duct—A hole formed in a concrete member to accommodate a tendon for post-tensioning; a pipe or runway for electric, telephone, or other utilities.

Ductility—That property of a material by virtue of which it may undergo large permanent deformation without rupture.

Dummy joint—See **Groove joint**.

Dunagan analysis—A method of separating the ingredients of freshly mixed concrete or mortar to determine the proportions of the mixture.

Durability—The ability of concrete to resist weathering action, chemical attack, abrasion, and other conditions of service.

Dusting—The development of a powdered material at the surface of hardened concrete.

Dynamic analysis—Analysis of stresses in framing as functions of displacement under transient loading.

Dynamic load—A load which is variable, i.e., not static, such as a moving live load, earthquake, or wind.

Dynamic loading—Loading from units (particularly machinery) which, by virtue of their movement or vibration, impose stresses in excess of those imposed by their dead load.

Dynamic modulus of elasticity—The modulus of elasticity computed from the size, weight, shape, and fundamental frequency of vibration of a concrete test specimen, or from pulse velocity.

E

Early strength—Strength of concrete or mortar usually as developed at various times during the first 72 hrs after placement.

Earth pigments—The class of pigments which are produced by physical processing of materials mined directly from the earth; also frequently termed natural or mineral pigments or colors.

Eccentric tendon—A prestressing tendon which follows a trajectory not coincident with the gravity axis of the concrete member.

Edge-bar reinforcement—Tension steel sometimes used to strengthen otherwise inadequate edges in a slab, without resorting to edge thickening.

Edge beam—A stiffening beam at the edge of a slab.

Edge form—Formwork used to limit the horizontal spread of fresh concrete on flat surfaces such as pavements or floors.

Edger—A finishing tool used on the edges of fresh concrete to provide a rounded corner.

Effective area of concrete—Area of a section assumed to be active in resisting the applied stresses; the area of a section which lies between the centroid of the tension reinforcement and the compression face of the flexural member.

Effective area of reinforcement—The area obtained by multiplying the right cross-sectional area of the metal reinforcement by the cosine of the angle between its direction and the direction for which its effectiveness is considered.

Effective area of reinforcement in diagonal bands—The area obtained by multiplying the normal cross-sectional area of the reinforcement by the cosine of the angle at which the band is inclined to the direction for which its effectiveness is considered.

Effective depth—Depth of a beam or slab section measured from the compression face to the centroid of the tensile reinforcement.

Effective flange width—Width of slab adjoining a beam stem where the slab is assumed to function as the flange element of a T-beam section.

Effective prestress—The stress remaining in concrete due to prestressing after all losses have occurred, excluding the effect of superimposed loads and the weight of the member; the stress remaining in the tendons after all losses have occurred excluding effects of dead load and superimposed load.

Effective span—The lesser of the two following distances: (a) the distance between centers of supports; (b) the clear distance between supports plus the effective depth of the beam or slab.

Effective stress—In prestressed concrete, the stress remaining in the tendons after all losses of the prestressing load have occurred.

Effective width of slab—That part of the width of a slab taken into account when designing T- or L-beams.

Efflorescence—A deposit of salts, usually white, formed on a surface, the substance having emerged in solution from within concrete or masonry and deposited by evaporation.

Elastic design—A method of analysis in which the design of a member is based on a linear stress-strain relationship and corresponding limiting elastic properties of the material.

Elastic limit—The limit of stress beyond which the strain is not wholly recoverable.

Elastic loss—In prestressed concrete, the reduction in prestressing load resulting from the elastic shortening of the member.

Elastic modulus—See **Modulus of elasticity**.

Elastic shortening—In prestressed concrete, the shortening of a member which occurs immediately on the application of forces induced by prestressing.

Elasticity—That property of a material by virtue of which it tends to recover its original size and shape after deformation.

Electrolysis—Production of chemical changes by the passage of current through an electrolyte.

Electrolyte—A conducting medium in which the flow of current is accompanied by movement of matter; usually an aqueous solution.

Elephant trunk—An articulated tube or chute used in concrete placement. (See also **Dropchute** and **Tremie**.)

Elongated piece (of aggregate)—Particle of aggregate for which the ratio of the length to the width of its circumscribing rectangular prism is greater than a specified value. (See also **Flat piece**.)

Embedment length—The length of embedded reinforcement provided beyond a critical section.

Embedment length equivalent—The length of embedded reinforcement which can develop the same stress as that which can be developed by a hook or mechanical anchorage.

Emery—A rock consisting essentially of an intercrystalline mixture of corundum and magnetite or hematite; a manufactured aggregate composed of emery used to produce a wear and slip resistant concrete floor surface. (See also **Dry shake**.)

Encastré—The end fixing of a built-in beam.

Enclosure wall—A nonload-bearing wall intended only to enclose space.

Encrustation—See **Incrustation**.

End anchorage—

1. Length of reinforcement, or a mechanical anchor, or a hook, or combination thereof, beyond the point of nominal zero stress in the reinforcement of cast-in-place concrete;
2. Mechanical device to transmit prestressing force to the concrete in a post-tensioned member.

See also **Anchorage.**

End block—An enlarged end section of a member designed to reduce anchorage stresses to allowable values.

Endothermic reaction—A chemical reaction which occurs with the absorption of heat.

Entrained air—Microscopic air bubbles intentionally incorporated in mortar or concrete during mixing, usually by use of a surface-active agent; typically between 10 and 1000 μm in diameter and spherical or nearly so. (See also **Air entrainment**.)

Entrapped air—Air voids in concrete which are not purposely entrained and which are significantly larger and less useful than those of entrained air, 1 mm or larger in size.

Epoxy concrete—A mixture of epoxy resin, catalyst, fine aggregate, and coarse aggregate. (See also **Epoxy mortar, Epoxy resins,** and **Polymer concrete**.)

Epoxy mortar—A mixture of epoxy resin, catalyst, and fine aggregate. (See also **Epoxy resins**.)

Epoxy resins—A class of organic chemical bonding systems used in the preparation of special coatings or adhesives for concrete or as binders in epoxy resin mortars and concretes.

Equivalent rectangular stress distribution—An assumption of uniform stress on compression side of neutral axis in strength method of design to determine flexural capacity.

Erosion—Progressive disintegration of a solid by the abrasive or cavitation action of gases, fluids, or solids in motion. (See also **Abrasion resistance** and **Cavitation damage**.)

Ettringite—A mineral, high sulfate calcium sulfoaluminate ($3\ CaO \cdot Al_2O_3 . 3\ CaSO_4 \cdot 30\text{–}32\ H_2O$) also written as $Ca_6\ [Al(OH)_6]_2 \cdot 24\ H_2O\ [(SO_4)_3 \cdot 1\frac{1}{2}\ H_2O]$; occurring in nature or formed by sulfate attack on mortar and concrete; the product of the principal expansion-producing reaction in expansive cements; designated as "cement bacillus" in older literature.

Evaporable water—Water in set cement paste present in capillaries or held by surface forces; measured as that removable by drying under specified conditions. (See also **Nonevaporable water**.)

Evaporation retardant—A long-chain organic material such as cetyl alcohol which when spread on a water film on the surface of concrete retards the evaporation of bleeding water.

Exfoliation—Disintegration occurring by peeling off in successive layers; swelling up and opening into leaves or plates like a partly opened book.

Exothermic reaction—A chemical reaction which occurs with the evolution of heat.

Expanded blast-furnace slag—The lightweight cellular material obtained by controlled processing of molten blast-furnace slag with water, or water and other agents, such as steam or compressed air or both. (See also **Blast-furnace slag**.)

Expanded metal lath—A metal network, often used as reinforcement in concrete or mortar construction, formed by suitably stamping or cutting sheet metal and stretching it to form open meshes, usually of diamond shape. (See also **Diamond mesh**.)

Expanded shale (clay or **slate)**—Lightweight vesicular aggregate obtained by firing suitable raw materials in a kiln or on a sintering grate under controlled conditions.

Expanding cement—See **Expansive cement**.

Expansion—See **Contraction**.

Expansion joint—A separation between adjoining parts of a concrete structure which is provided to allow small relative movements such as those caused by thermal changes to occur independently.

Expansion sleeve—A tubular metal covering for a dowel bar to allow its free longitudinal movement at a joint.

Expansive cement (general)—A cement which when mixed with water forms a paste that, after setting, tends to increase in volume to a significantly greater degree than portland cement paste; used to compensate for volume decrease due to shrinkage or to induce tensile stress in reinforcement (post-tensioning).

Expansive Cement, Type K—A mixture of portland cement, anhydrous tetracalcium trialuminate sulfate ($C_4A_3\bar{S}$), calcium sulfate ($CaSO_4$), and lime (CaO); the $C_4A_3\bar{S}$ is a constituent of a separately burned clinker that is interground with portland cement or alternately, it may be formed simultaneously with the portland cement clinker compounds during the burning process.

Expansive cement, Type M—Interground or blended mixtures of portland cement, calcium-aluminate cement, and calcium sulfate suitably proportioned.

Expansive cement, Type S—A portland cement containing a large computed tricalcium aluminate (C_3A) content and an amount of calcium sulfate above the usual amount found in portland cement.

Expansive-cement concrete (mortar or **grout)**—A concrete (mortar or grout) made with expansive cement.

Expansive component—The portion of an expansive cement which is responsible for the expansion, generally one of several anhydrous calcium aluminate or sulfoaluminate compounds and a source of sulfate, with or without free lime, (CaO). The expansive component may be produced separately and later ground or blended with a normal portland cement clinker, in other instances, produced by firing in a kiln with the constituents of portland cement.

Exposed-aggregate finish—A decorative finish for concrete work achieved by removing, generally before the concrete has fully hardened, the outer skin of mortar and exposing the coarse aggregate.

Exposed concrete—Concrete surfaces formed so as to yield an acceptable texture and finish for permanent exposure to view. (See also **Architectural concrete**.)

Extender—A finely divided inert mineral added to provide economical bulk in paints, synthetic resins and adhesives, or other products.

Extensibility—The maximum tensile strain that hardened cement paste, mortar, or concrete can sustain before cracking occurs.

Extension device—Any device, other than an adjustment screw, used to obtain vertical adjustment of shoring towers.

Exterior panel—In a flat slab, a panel having at least one edge which is not in common with another panel.

External vibrator—See **Vibration**.

Extreme compression fiber—Farthest fiber from the neutral axis on the compression side of a member subjected to bending.

Extreme tension fiber—Farthest fiber from the neutral axis on the tension side of a member subjected to bending.

Exudation—A liquid or viscous gel-like material discharged through a pore, crack, or opening in the surface of concrete.

F

Factor of safety—The ratio of the ultimate strength (or yield strength) of a material to the working stress assumed in the design (stress factor of safety); may also be expressed as the ratio of load, moment, or shear of structural member at the ultimate to that at the working level (load factor of safety).

Fair face concrete—A concrete surface which, on completion of the forming process, requires no further (concrete) treatment other than curing. (See also **Architectural concrete**.)

False header—See **Header**.

False set—The rapid development of rigidity in a freshly mixed portland cement paste, mortar, or concrete without the evolution of much heat, which rigidity can be dispelled and plasticity regained by further mixing without addition of water; premature stiffening, hesitation

Fal

set, early stiffening, and rubber set are terms referring to the same phenomenon, but false set is the preferred designation.

Falsework—The temporary structure erected to support work in the process of construction; composed of shoring or vertical posting, formwork for beams and slabs, and lateral bracing. (See also **Centering.**)

Fascia—A flat member or band at the surface of a building or the edge beam of a bridge; exposed eave of a building; often inappropriately called facia.

Fat concrete—Concrete containing a relatively large amount of plastic and cohesive mortar.

Fatigue—The weakening of a material caused by repeated or alternating loads.

Fatigue failure—The phenomenon of rupture of a material, when subjected to repeated loadings, at a stress substantially less than the ultimate static strength.

Fatigue strength—The greatest stress which can be sustained for a given number of stress cycles without failure.

Faulting—Differential vertical displacement of a slab or other member adjacent to a joint or crack.

Feather edge—A wood or metal tool having a beveled edge; used to straighten re-entrant angles in finish plaster coat; also edge of a concrete or mortar placement such as a patch or topping that is beveled at an acute angle.

Feed wheel—Material distributor or regulator in certain types of shotcrete equipment.

Felite—A name used by Tornebohm (1897) to identify one form of dicalcium silicate ($2CaO \cdot SiO_2$), one of the crystalline components of portland cement clinker. (See also **Alite, Belite,** and **Celite.**)

Ferrocement—A composite hydraulic structural material comprising thin sections consisting of cement mortar reinforced by a number of very closely spaced layers of steel wire mesh.

Fiber-reinforced concrete—See **Concrete, fibrous**.

Field bending—Bending of reinforcing bars on the job rather than in a fabricating shop.

Field concrete—See **Concrete, field**.

Field-cured cylinders—Test cylinders cured as nearly as practicable in the same manner as the concrete in the structure to indicate when supporting forms may be removed, additional construction loads may be imposed, or the structure may be placed in service.

Filler—

1. Finely divided inert material such as pulverized limestone, silica, or colloidal substances sometimes added to portland cement paint or other materials to reduce shrinkage, improve workability, or act as an extender.
2. Material used to fill an opening in a form.

Fillet—A concave junction formed where two surfaces meet. (See also **Chamfer strip.**)

Fin—A narrow linear projection on a formed concrete surface, resulting from mortar flowing out between spaces in the formwork.

Final prestress—See **Final stress**.

Final set—A degree of stiffening of a mixture of cement and water greater than initial set, generally stated as an empirical value indicating the time in hours and minutes required for a cement paste to stiffen sufficiently to resist to an established degree, the penetration of a weighted test needle; also applicable to concrete and mortar mixtures with use of suitable test procedures. (See also **Initial set.**)

Final setting time—The time required for a freshly mixed cement paste, mortar, or concrete to achieve final set. (See also **Initial setting time.**)

Final stress—In prestressed concrete, the stress which exists after substantially all losses have occurred.

Fine aggregate—Aggregate passing the ⅜-in. (9.5-mm) sieve and almost entirely passing the No. 4 (4.75-mm) sieve and predominantly retained on the No. 200 (75-μm) sieve; or that portion of an aggregate passing the No. 4 (4.75-mm) sieve and predominantly retained on the No. 200 (75-μm) sieve. (See also **Aggregate** and **Sand.**)

Fine grained soil—Soil in which the smaller grain sizes predominate, such as fine sand, silt, and clay.

Fineness—A measure of particle size.

Fineness modulus—A factor obtained by adding the total percentages by weight of an aggregate sample retained on each of a specified series of sieves, and dividing the sum by 100; in the United States the standard sieve sizes are No. 100 (150 μm), No. 50 (300 μm), No. 30 (600 μm), No. 16 (1.18 mm), No. 8 (2.36 mm) and No. 4 (4.75 mm), and ⅜ in. (9.5 mm), ¾ in. (19 mm), 1½ in. (38.1 mm), 3 in. (75 mm), and 6 in. (150 mm).

Finish—The texture of a surface after compacting and finishing operations have been performed.

Finish coat—Final thin coat of shotcrete preparatory to hand finishing; also exposed coat of plaster and stucco.

Finish grinding—The final grinding of clinker into cement, with calcium sulfate in the form of gypsum or anhydrite generally being added; the final grinding operation required for a finished concrete surface, e.g., bump cutting of pavement, fin removal from structural concrete, terrazzo floor grinding.

Finishing—Leveling, smoothing, compacting, and otherwise treating surfaces of fresh or recently placed concrete or mortar to produce desired appearance and service. (See also **Float** and **Trowel.**)

Finishing machine—A power-operated machine used to give the desired surface texture to a concrete slab.

Fire clay—An earthy or stony mineral aggregate which has as the essential constituent hydrous silicates of aluminum with or without free silica, plastic when sufficiently pulverized and wetted, rigid when subsequently dried, and of suitable refractoriness for use in commercial refractory products.

Fire resistance—The property of a material or assembly to withstand fire or give protection from it; as applied to elements of buildings, it is characterized by the ability to confine a fire or to continue to perform a given structural function, or both.

Fired strength—The compressive or flexural strength of refractory concrete determined upon cooling after first firing to a specified temperature for a specified time.

Fired unit weight—The unit weight of refractory concrete, upon cooling, after having been exposed to a specified firing temperature for a specified time.

Fishtail—A wedge-shaped piece of wood used as part of the support form between tapered pans in concrete joist construction.

Flame photometer—An instrument used to determine elements (especially sodium and potassium in portland cement) by the color intensity of their unique flame spectra resulting from introducing a solution of a compound of the element into a flame. (Also known as **Flame spectrophotometer.**)

Flash coat—A light coat of shotcrete used to cover minor blemishes on a concrete surface.

Flash set—The rapid development of rigidity in a freshly mixed portland cement paste, mortar, or concrete, usually with the evolution of considerable heat, which rigidity cannot be dispelled nor can the plasticity be regained by further mixing without addition of water; also referred to as quick set or grab set.

Flashing—A thin impermeable sheet, narrow in comparison with its length, installed as a "waterproof" cover over exposed joints, at roof valleys, hips, roof parapets, or intersections of roof and chimney.

Flat jack—A hydraulic jack consisting of light gage metal bent and welded to a flat shape which expands under internal pressure.

Flat piece (of aggregate)—One in which the ratio of the width to thickness of its circumscribing rectangular prism is greater than a specified value. (See also **Elongated piece (of aggregate**.)

Flat plate—A flat slab without column capitals or drop panels. (See also **Flat slab**.)

Flat slab—A concrete slab reinforced in two or more directions, generally without beams or girders to transfer the loads to supporting members. (See also **Flat plate**.)

Flexible pavement—A pavement structure which maintains intimate contact with and distributes loads to the subgrade and depends on aggregate interlock, particle friction, and cohesion for stability; cementing agents, where used, are generally bituminous materials as contrasted to portland cement in the case of rigid pavement. (See also **Rigid pavement**.)

Flexural bond—In prestressed concrete, the stress between the concrete and the tendon which results from the application of external load.

Flexural rigidity—A measure of stiffness of a member, indicated by the product of modulus of elasticity and moment of inertia divided by the length of the member.

Flexural strength—A property of a material or structural member that indicates its ability to resist failure in bending. (See also **Modulus of rupture**.)

Flint—A variety of chert. (See also **Chert**.)

Float—A tool (not a darby), usually of wood, aluminum, or magnesium, used in finishing operations to impart a relatively even but still open texture to an unformed fresh concrete surface.

Float finish—A rather rough concrete surface texture obtained by finishing with a float.

Floating—The operation of finishing a fresh concrete or mortar surface by use of a float, preceding troweling when that is the final finish.

Flow—

1. Time dependent irrecoverable deformation. (See **Rheology**.)

2. A measure of the consistency of freshly mixed concrete, mortar, or cement paste in terms of the increase in diameter of a molded truncated cone specimen after jigging a specified number of times.

Flow cone—A device for measurement of grout consistency in which a predetermined volume of grout is permitted to escape through a precisely sized orifice, the time of efflux (flow factor) being used as the indication of consistency; also the mold used to prepare a specimen for the flow test.

Flow factor—See **Flow cone**.

Flow promoter—Substance added to coating to enhance brushability, flow, and leveling.

Flow table—A jigging device used in making flow tests for consistency of cement paste, mortar, or concrete. (See also **Flow (2)**.)

Flow trough—A sloping trough used to convey concrete by gravity flow from a transit mix truck, or a receiving hopper to the point of placement. (See also **Chute**.)

Fluidifier—An admixture employed in grout to decrease the flow factor without changing water content.

Fluosilicate—Magnesium or zinc silico-fluoride used to prepare aqueous solutions sometimes applied to concrete as surface-hardening agents.

Flush water—See **Wash water**.

Fly ash—The finely divided residue resulting from the combustion of ground or powdered coal and which is transported from the firebox through the boiler by flue gases; known in UK as **Pulverized fuel ash (pfa)**.

Foamed blast-furnace slag—See **Expanded blast-furnace slag**.

Foamed concrete—See **Concrete, foamed**.

Fog curing—

1. Storage of concrete in a moist room in which the desired high humidity is achieved by the atomization of fresh water. (See also **Moist room**.)

2. Application of atomized fresh water to concrete, stucco, mortar, or plaster.

Folded plate—

1. A framing assembly composed of sloping slabs in a hipped or gabled arrangement.

2. Prismatic shell with open polygonal section.

Footing—That portion of the foundation of a structure which spreads and transmits load directly to the piles, or to the soil or supporting grillage.

Form—A temporary structure or mold for the support of concrete while it is setting and gaining sufficient strength to be self supporting. (See also **Formwork**.)

Form anchor—Device used to secure formwork to previously placed concrete of adequate strength; the device is normally embedded in the concrete during placement.

Form coating—A liquid applied to usually interior formwork surfaces for a specific purpose, usually to promote easy release from the concrete, to preserve the form material or to retard set of the near-surface matrix for preparation of exposed-aggregate finishes.

Form hanger—Device used to support formwork from a structural framework; the dead load of forms, weight of concrete, and construction and impact loads must be supported.

Form insulation—Insulating material applied to outside of forms between studs and over the top in sufficient thickness and air tightness to conserve heat of hydration to maintain concrete at required temperatures in cold weather.

Form lining—Selected materials used to line the concreting face of formwork in order to impart a smooth or patterned finish to the concrete surface, to absorb moisture from the concrete, or to apply a set-retarding chemical to the formed surface.

Form oil—Oil applied to interior surface of formwork to promote easy release from the concrete when forms are removed.

Form pressure—Lateral pressure acting on vertical or inclined formed surfaces, resulting from the fluid-like behavior of the unhardened concrete confined by the forms.

Heavy-media separation—A method in which a liquid or suspension of given specific gravity is used to separate particles into a portion lighter than (those that float) and a portion heavier than (those that sink) the medium.

Heavyweight aggregate—See **Aggregate, heavyweight.**

Heavyweight concrete—See **High density concrete.**

Hematite—A mineral, iron oxide (Fe_2O_3), used as aggregate in high density concrete and in finely divided form as a red pigment in colored concrete.

Hemihydrate—A hydrate containing one-half molecule of water to one molecule of compound, the most commonly known hemihydrate is partially dehydrated gypsum (also known as **Plaster of Paris**), $CaSO_4 \cdot \frac{1}{2}H_2O$. (See also **Bassanite.**)

Hesitation set—See **False set.**

Hessian—See **Burlap.**

High alumina cement—See **Calcium-aluminate cement.**

High-bond bar—See **Deformed bar.**

High-density concrete—Concrete of exceptionally high density, usually obtained by use of heavyweight aggregates, used especially for radiation shielding.

High-discharge mixer—See **Inclined-axis mixer.**

High-early-strength cement—See **Cement, high-early-strength.**

High-early-strength concrete—Concrete which, through the use of high-early-strength cement or admixtures, is capable of attaining specified strength at an earlier age than normal concrete.

High-lift grouting—A technique in concrete masonry wall construction in which the grouting operation is delayed until the wall has been laid up to a full story height.

High pressure steam curing—See **Autoclave curing.**

High strength reinforcement—See **Reinforcement, high strength.**

High-strength steel—Steel with a high yield point, in the case of reinforcing bars 60,000 psi (414MPa) and greater. (See also **Reinforcement, high strength.**)

High temperature steam curing—See **Atmospheric-pressure steam curing,** and **Autoclave curing.**

Hinge joint—Any joint which permits hinge action with no appreciable separation of the adjacent members.

Hod—A portable trough for carrying mortar, brickes, etc., fixed crosswise on top of a pole and carried on the shoulder.

Holding-down bolt—See **Anchor bolt.**

Holding period—See **Presteaming period.**

Honeycomb—Voids left in concrete due to failure of the mortar to effectively fill the spaces among coarse aggregate particles.

Hook—A bend in the end of a reinforcing bar.

Hooked bar—A reinforcing bar with the end bent into a hook to provide anchorage.

Hooke's Law—The law, which holds practically for strains within the elastic limit, that the strain is proportional to the stress producing it. (See also **Proportional limit** and **Modulus of elasticity.**)

Hoop—A one-piece closed tie or continuously wound tie not less than #3 in size, the ends of which have a standard 135-deg bend with a ten-bar-diameter extension, that encloses the longitudinal reinforcement.

Horizontal-axis mixer—A concrete mixer of the revolving drum type in which the drum rotates about a horizontal axis.

Horizontal brace—See **Ledger.**

Horizontal shoring—See **Shoring, horizontal.**

Hot cement—Newly manufactured cement which has not had an opportunity to cool after burning and grinding of the component materials.

Hot face—The surface of a refractory section exposed to the source of heat.

Hot load test—A test for determining the resistance to deformation or shear of a refractory material when subjected to a specified compressive load at a specified temperature for a specified time.

Hoyer effect—In prestressed concrete, frictional forces which result from the tendency of the tendons to regain the diameter which they had before they were stressed.

Hydrate—A chemical combination of water with another compound or an element.

Hydrated lime—Calcium hydroxide, a dry powder obtained by treating quicklime with water.

Hydration—Formation of a compound by the combining of water with some other substance; in concrete, the chemical reaction between hydraulic cement and water.

Hydraulic cement—A cement that sets and hardens by chemical interaction with water and that is capable of doing so under water.

Hydraulic hydrated lime—The hydrated dry cementitious product obtained by calcining a limestone containing silica and alumina to a temperature short of incipient fusion so as to form sufficient free calcium oxide to permit hydration and at the same time leaving unhydrated sufficient calcium silicates to give the dry powder its hydraulic properties.

I

Ignition loss—See **Loss on ignition.**

Ilmenite—A mineral, iron titanate ($FeTiO_3$), which in pure or impure form is commonly used as aggregate in high density concrete.

Impending slough—The consistency obtained with shotcrete containing the maximum amount of water that can be used without flow or sag after placement.

Inclined-axis mixer—A truck with revolving drum which rotates about an axis inclined to the bed of the truck chassis.

Incrustation—A crust or coating, generally hard, formed on the surface of concrete or masonry construction or on aggregate particles.

Indented wire—Wire having machine-made surface indentations intended to improve bond; depending on type of wire, may be used for either concrete reinforcement or pretensioning tendons.

Industrialized building—The integration of planning, design, programming, manufacturing, site operations, scheduling, financing and management into a disciplined method of mechanized production of buildings, sometimes called **Systems building.**

Inelastic behavior—See **Plastic deformation.**

Infrared spectroscopy—The use of a spectrophotometer for determination of infrared absorption spectra (2.5 to 18 μm wave lengths) of materials; used for detection, determination, and identification especially of organic materials.

Initial drying shrinkage—The difference between the length of a specimen (molded and cured under stated conditions) and its length when first dried to constant length, expressed as a percentage of the moist length.

Initial prestress—The prestressing stress (or force) applied to the concrete at the time of stressing.

Initial set—A degree of stiffening of a mixture of cement and water less than final set, generally stated as an empirical value indicating the time in hours and minutes required for cement paste to stiffen sufficiently to resist to an established degree, the penetration of a weighted test needle; also applicable to concrete or mortar with use of suitable test procedures. (See also **Final set.**)

Initial setting time—The time required for a freshly mixed cement paste, mortar or concrete to achieve initial set. (See also **Final setting time.**)

Initial stresses—The stresses occurring in prestressed concrete members before any losses occur.

Initial tangent modulus—See **Modulus of elasticity.**

In-situ concrete—See **Concrete, in-situ.**

Insoluble residue—The portion of a cement or aggregate that is not soluble in dilute hydrochloric acid of stated concentration.

Insulating concrete—Concrete having low thermal conductivity; used as thermal insulation.

Internal vibration—See **Vibration.**

I-section—Beam cross section consisting of top and bottom flanges connected by a vertical web.

Isolation joint—A separation between adjoining parts of a concrete structure, usually a vertical plane, at a designed location such as to interfere least with performance of the structure, yet such as to allow relative movement and avoid formation of cracks elsewhere in the concrete and through which all or part of the bonded reinforcement is interrupted. (See also **Contraction joint.**)

Isotropy—The behavior of a medium having the same properties in all directions.

J

Jack—A mechanical device used to apply force to prestressing tendons, adjust elevation of forms or form supports, and raise objects small distances.

Jack shore—Telescoping, or otherwise adjustable, single-post metal shore.

Jacking device—The device used to stress the tendons for prestressed concrete; also, a device for raising a vertical slipform.

Jacking force—In prestressed concrete, the temporary force exerted by the device which introduces tension into the tendons.

Jacking stress—The maximum stress occurring in a prestressed tendon during stressing.

Jaw crusher—A machine having two inclined jaws, one or both being actuated by a reciprocating motion so that the charge is repeatedly "nipped" between the jaws.

Jitterbug—A grate tamper for pushing coarse aggregate slightly below the surface of a slab to facilitate finishing.

Joint, construction—See **Construction joint.**

Joint, contraction—See **Contraction joint.**

Joint, expansion—See **Expansion joint.**

Joint filler—Compressible material used to fill a joint to prevent the infiltration of debris and to provide support for sealants.

Joint sealant—Compressible material used to exclude water and solid foreign materials from joints.

Jointer (concrete)—A metal tool about 6 in. (150 mm) long and from 2 to 4½ in. (50 to 100 mm) wide and having shallow, medium, or deep bits (cutting edges) ranging from $^3/_{16}$ in. to ¾ in. (5 to 20 mm) or deeper used to cut a joint partly through fresh concrete.

Joist—A comparatively narrow beam, used in closely spaced arrangements to support floor or roof slabs which require no reinforcement except that required for temperature and shrinkage stresses; also a horizontal structural member such as that which supports deck form sheathing. (See also **Beam.**)

Jumbo—Traveling support for forms, commonly used in tunnel work.

K

Kaolin—A rock, generally white, consisting primarily of clay minerals of the kaolinite group, composed principally of hydrous aluminum silicate, of low iron content, used as raw material in the manufacture of white cement.

Kaolinite—A common clay mineral having the general formula Al_2 (Si_2O_5) (OH_4), the primary constituent of kaolin.

Keene's cement—A cement composed of finely ground, anhydrous, calcined gypsum, the set of which is accelerated by the addition of other materials.

Kelly ball—An apparatus used for indicating the consistency of fresh concrete, consisting of a cylinder 6 in. (150 mm) in diameter with a hemispherically shaped bottom and handle weighing 30 lb (14 kg) and a stirrup to guide the handle and serve as a reference for measuring depth of penetration. (See also **Ball test.**)

Kelly ball test—See **Ball test** and **Kelly ball.**

Kerb form; Kerb tool—See **Curb form** and **Curb tool.**

Kerf—To cut or notch, as a beam, transversely along the underside to curve it; also a cut or notch in a member such as a rustication strip to avoid damage from swelling of the wood and to permit easier removal.

Kern area—The area within a geometric shape in which a compressive force may be applied without tensile stresses resulting in any of the extreme fibers of the section.

Kern distance—The ratio of the section modulus of the cross section about the axis perpendicular to the kern distance, divided by the area of the cross section.

Key—See **Keyway.**

Keyed, Keying—Fastened or fixed in position in a notch or other recess.

Keyway—A recess or groove in one lift or placement of concrete which is filled with concrete of the next lift, giving shear strength to the joint.

Kick strip—See **Kicker.**

Kicker—A wood block or board attached to a formwork member in a building frame or formwork to make the structure more stable; in formwork it acts as a haunch. (See also **Stub wall.**)

Kiln—A furnace or oven for drying, charring, hardening, baking, calcining, sintering, or burning various materials. (See also **Steam-curing room.**)

Kiln, Cement—A kiln in which the ground and proportioned raw mix is dried, calcined, and burned into clinker at a temperature of 2600 to 3000 F (1420 to 1650 C); can be of the rotary, shaft, fluid-bed, or traveling grate type; fuel may be coal, oil, or gas.

Kip

Kip—1000 lb force, equals 4448.222 newtons.

Knee brace—Brace between horizontal and vertical members in a building frame or formwork to make the structure more stable; in formwork it acts as a haunch.

L

Lacing—Horizontal bracing between shoring members.

Lagging—Heavy sheathing used as in underground work to withstand earth pressure. (See also **Sheathing.**)

Laitance—A layer of weak and nondurable material containing cement and fines from aggregates, brought by bleeding water to the top of overwet concrete, the amount of which is generally increased by overworking or overmanipulating concrete at the surface by improper finishing or by job traffic.

Lap—The length by which one bar or sheet of fabric reinforcement overlaps another.

Lap splice—A connection of reinforcing steel made by lapping the ends of the bars.

Lapping (reinforcing steel)—The overlapping of reinforcing steel bars, welded wire fabric, or expanded metal so that there may be continuity of stress in the reinforcing when the concrete member is subjected to flexural or tensile loading.

Larnite—A mineral; beta dicalcium silicate (Ca_2SiO_4); occurs naturally at Scawt Hill, Northern Ireland, and artificially in slags and as a major constituent of portland cement.

Lateral reinforcement—See **Reinforcement, lateral.**

Latex—A water emulsion of a synthetic rubber or plastic obtained by polymerization and used especially in coatings and adhesives.

Layer—See **Course.**

L-beam—A beam whose section has the form of an inverted L, usually occurring in the edge of a floor, of which a part forms the top flange of the beam.

L-column—The portion of a precast concrete frame, composed of the column, the haunch, and part of the girder.

Leaf—See **Wythe.**

Lean concrete—Concrete of low cement content.

Ledger—An L-shaped horizontal member that supports other permanent or temporary structural members. (See also **Beam.**)

Lever arm—In a structural member, the distance from the center of the tensile reinforcement to the center of action of the compression.

L-head—The top of a shore formed with a braced horizontal member projecting from one side forming an inverted L-shaped assembly.

Lift—The concrete placed between two consecutive horizontal construction joints, usually consisting of several layers or courses.

Lift joint—Surface at which two successive lifts meet.

Lift slab—A method of concrete construction in which floor and roof slabs are cast on or at ground level and hoisted into position by jacking; also a slab which is a component of such construction.

Lifts (or **Tiers**)—The number of frames of scaffolding erected one above each other in a vertical direction.

Lightweight aggregate—See **Aggregate, lightweight.**

Lightweight concrete—Concrete of substantially lower unit weight than that made using gravel or crushed stone aggregates.

Lime—Specifically, calcium oxide (CaO); also, loosely, a general term for the various chemical and physical forms of quicklime, hydrated lime, and hydraulic hydrated lime.

Limit design—A method of proportioning reinforced concrete members based on calculations of their strength. (See also **Strength design method.**)

Limonite—An iron ore composed of a mixture of hydrated ferric oxides; occasionally used in high density concrete because of its high density and water content which contribute to its effectiveness in radiation shielding. (See also **Brown oxide.**)

Linear prestressing—Prestressing as applied to linear members, such as beams, columns, etc.

Linear transformation—The method of altering the trajectory of the prestressing tendon in any statically indeterminate prestressed structure by changing the location of the tendon at one or more interior supports without altering its position at the end supports and without changing the basic shape of the trajectory between any supports; linear transformation does not change the location of trajectory of the pressure line.

Linear traverse method—Determination of the volumetric composition of a solid by integrating the distance traversed across areas of each component along a line or along regularly spaced lines in one or more planes intersecting a sample of the solid; frequently employed to determine characteristics of the air-void system in hardened concrete by microscopical examination along a series of traverse lines on finely ground sections of the concrete; sometimes called the **Rosiwal method.**

Lining—Any sheet, plate, or layer of material attached directly to the inside face of formwork to improve or alter the surface texture and quality of the finished concrete. (See also **Form lining.**)

Lintel—A horizontal supporting member above an opening such as a window or a door.

Liquid limit—Water content, expressed as a percentage of the dry weight of the soil at which the soil passes from the plastic to the liquid state under standard test conditions. (See also **Atterberg limits.**)

Liquid-volume measurement—Measurement of grout on the basis of the total volume of solid and liquid constituents.

Live load—Any load that is not permanently applied to a structure.

Load, service—See **Service dead load** and **Service live load.**

Load binder—A device used to tighten chains holding loads in place on a truck bed.

Load-factor—A factor by which a service load is multiplied to determine a design load. (See also **Phi (ϕ) factor.**)

Load-bearing wall—A wall designed and built to carry superimposed vertical and shear loads as opposed to nonload-bearing walls.

Load-transfer assembly—Most commonly, the unit (basket or plate) designed to support or link dowel bars during concreting operations so as to hold them in place, in the desired alignment.

Loading hopper—A hopper in which concrete or other free flowing material is placed for loading by gravity into buggies or other conveyances for transport to the forms or to other place of processing, use, or storage.

Locking device—A device used to secure a cross brace in scaffolding to the frame or panel.

Long column—See **Column, long.**

Longitudinal bar—See **Longitudinal reinforcement.**

Longitudinal joint—A joint parallel to the long dimension of a structure or pavement.

Longitudinal reinforcement—Reinforcement essentially parallel to the long axis of a concrete member or pavement.

Los Angeles Abrasion Test—Test for abrasion resistance of concrete aggregates.

Loss of prestress—The reduction in the prestressing force which results from the combined effects of strains in the concrete and steel, including slip at anchorage, relaxation of steel stress, frictional loss due to curvature in the tendons and the effects of elastic shortening, creep and shrinkage of the concrete.

Loss on ignition—The percentage loss in weight of a sample ignited to constant weight at a specified temperature, usually 900-1000 C.

Low-alkali cement—See **Cement, low-alkali.**

Low-density concrete—Concrete having an oven-dry unit weight of less than 50 pcf (800 kg/m^3).

Low-heat cement—See **Cement, low-heat.**

Low-lift grouting—The common and simple method of unifying concrete masonry, in which the wall sections are built to a height of not more than 4 ft. (1.2 m) before the cells of the masonry units are filled with grout.

Low-pressure steam curing—See **Atmospheric-pressure steam curing.**

L-shore—A shore with an L-head. (See also **L-head.**)

M

Macroscopic—See **Megascopic.**

Magnetite—A mineral, ferrous ferric oxide ($FeFe_2O_4$); the principal constituent of magnetic black iron ore; specific gravity about 5.2 and Mohs hardness about 6; used as an aggregate in high density concrete.

Main bar—See **Main reinforcement.**

Main reinforcement—Steel reinforcement designed to resist stresses resulting from design loads and moments, as opposed to reinforcement intended to resist secondary stresses.

Manual batcher—See **Batcher.**

Manufactured sand—See **Sand.**

Map cracking—See **Crazing.**

Marl—Calcareous clay, usually containing from 35 to 65 percent calcium carbonate ($CaCO_3$), found in the bottoms of shallow lakes, swamps, or extinct fresh-water basins.

Masonry—Construction composed of shaped or molded units, usually small enough to be handled by one man and composed of stone, ceramic brick or tile, concrete, glass, adobe, or the like; sometimes used to designate cast-in-place concrete.

Masonry cement—See **Cement, masonry.**

Masonry filler unit—Masonry unit used to fill in between joists or beams to provide a platform for a cast-in-place concrete slab.

Masonry mortar—Mortar used in masonry structures. (See also **Cement, masonry** and **Mortar.**)

Masonry unit—A construction unit in masonry. (See also **Block.**)

Mass concrete—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 curing—Adiabatic curing in sealed containers.

Mat—See **Bar mat.**

Mat foundation—A continuous footing supporting an array of columns in several rows in each direction, having a slab-like shape with or without depressions or openings, covering an area at least 75 percent of the total area within the outer limits of the assembly. (See also **Raft foundation.**)

Material hose—See **Delivery hose.**

Matrix—In the case of mortar, the cement paste in which the fine aggregate particles are embedded; in the case of concrete, the mortar in which the coarse aggregate particles are embedded.

Maximum size of aggregate—In specifications for or descriptions of aggregate, the smallest sieve opening through which the entire amount of aggregate is *required* to pass. (See also **Nominal maximum size of aggregate.**)

Maximum-temperature period—A time interval throughout which the maximum temperature is held constant in an autoclave or steam-curing room.

Mechanical analysis—The process of determining particle-size distribution of an aggregate. (See also **Sieve analysis.**)

Mechanical anchorage—Any mechanical device capable of developing the strength of the reinforcement without damage to the concrete.

Mechanical bond—In general concrete construction, the physical interlock between cement paste and aggregate, or between concrete and reinforcement (specifically, the sliding resistance of an embedded bar and not the adhesive resistance). In plastering, the physical keying of a plaster coat to: (a) another, (b) to the plaster base by means of plaster keys to the lath, or (c) through interlock with adjacent plaster casts created by means of scratching or cross raking.

Megascopic—Visible to the unaided eye.

Melilite—A group of minerals ranging from the calcium magnesium silicate (ackermanite) to the calcium aluminate silicate (gehlenite) that occur as crystals in blast-furnace slag. (See also **Merwinite.**)

Melt—The molten portion of the raw material mass during the burning of cement clinker, firing of lightweight aggregates, or expanding of blast-furnace slags.

Membrane curing—A process that involves either liquid sealing compound (e.g., bituminous and paraffinic emulsions, coal tar cut-backs, pigmented and nonpigmented resin suspensions, or suspensions of wax and drying oil) or nonliquid protective coating (e.g., sheet plastics or "waterproof" paper), both of which types function as films to restrict evaporation of mixing water from the fresh concrete surface.

Membrane theory—A theory of design for thin shells, based on the premise that a shell cannot resist bending because it deflects; the only stresses that exist, therefore, in any section are shear stress and direct compression or tension.

Merwinite—One of the principal crystalline phases found in blast-furnace slags; chemical formula is $3CaO \cdot MgO \cdot 2SiO_2$, crystal system is monoclinic, and specific gravity is 3.15. (See also **Melilite.**)

Mesh—See **Welded-wire fabric.**

Mesh reinforcement—See **Welded-wire fabric reinforcement.**

Micron—Name formerly used for micrometre (μm); a unit of length, one-thousandth of a millimeter or one-millionth of a meter.

Microscopic—Discernible only with the aid of a microscope.

Microcracks—Microscopic cracks within concrete.

Middle strip—In flat slab framing, the slab portion which occupies the middle half of the span between columns. (See also **Column strip.**)

Mill scale—The oxide layer formed during the hot rolling of metals, such as that formed on hot-rolled reinforcing bars.

Mineral aggregate—Aggregate consisting essentially of inorganic nonmetallic materials.

Mix—The act or process of mixing; also mixture of materials, such as mortar or concrete.

Mix design—See **Proportioning.**

Mixer—A machine used for blending the constituents of concrete, grout, mortar, cement paste, or other mixture.

Mixer, batch—See **Batch mixer.**

Mixer, colloidal—A mixer designed to produce colloidal grout.

Mixer, horizontal shaft—A mixer having a stationary cylindrical mixing compartment, with the axis of the cylinder horizontal, and one or more rotating horizontal shafts to which mixing blades or paddles are attached.

Mixer, nontilting—A horizontally rotating drum mixer that charges, mixes, and discharges without tilting.

Mixer, open-top—A truck-mounted mixer consisting of a trough or a segment of a cylindrical mixing compartment within which paddles or blades rotate about the horizontal axis of the trough. (See also **Mixer, horizontal shaft.**)

Mixer, pan—See **Mixer, vertical shaft.**

Mixer, tilting—A rotating drum mixer that discharges by tilting the drum about a fixed or movable horizontal axis at right angles to the drum axis. The drum axis may be horizontal or inclined while charging and mixing.

Mixer, vertical shaft—A cylindrical or annular mixing compartment having an essentially level floor and containing one or more vertical rotating shafts to which blades or paddles are attached; the mixing compartment may be stationary or rotate about a vertical axis.

Mixer efficiency—The adequacy of a mixer in rendering a homogeneous product within a stated period; homogeneity is determinable by testing for relative differences in physical properties of samples extracted from different portions of a freshly mixed batch.

Mixing cycle—The time taken for a complete cycle in a batch mixer, i.e., the time elapsing between successive repetitions of the same operation (e.g., successive discharges of the mixer).

Mixing plant—See **Batch plant.**

Mixing speed—Rotation rate of a mixer drum or of the paddles in an open-top, pan, or trough mixer, when mixing a batch; expressed in revolutions per minute (rpm), or in peripheral feet per minute of a point on the circumference at maximum diameter.

Mixing time—The period during which the constituents of a batch of concrete are mixed by a mixer; for a stationary mixer, time is given in minutes from the completion of mixer charging until the beginning of discharge; for a truck mixer, time is given in total minutes at a specified mixing speed or expressed in terms of total revolutions at a specified mixing speed. (See also **Amount of mixing.**)

Mixing water—The water in freshly mixed sand-cement grout, mortar, or concrete, exclusive of any previously absorbed by the aggregate (e.g., water considered in the computation of the net water-cement ratio). (See also **Batched water** and **Surface moisture.**)

Mixture—The assembled, blended, commingled ingredients of mortar, concrete, or the like; or the proportions for their assembly.

Modular ratio—The ratio of modulus of elasticity of steel E_S to that of concrete E_c usually denoted by the symbol n.

Modulus of deformation—A concept of modulus of elasticity expressed as a function of two time variables; strain in loaded concrete as a function of the age at which the load is initially applied and of the length of time the load is sustained.

Modulus of elasticity—The ratio of normal stress to corresponding strain for tensile or compressive stresses below the proportional limit of the material; referred to as "elastic modulus of elasticity;" "Young's modulus," and "Young's modulus of elasticity;" denoted by the symbol E. (See also **Modulus of rigidity.**)

Note: Few materials conform to Hooke's law throughout the entire range of stress-strain relations; deviations there from are caused by inelastic behavior. If the deviations are significant, the slope of the tangent to the stress-strain curve at the origin, the slope of the tangent to the stress-strain curve at any given stress, the slope of the secant drawn from the origin to any specified point on the stress-strain curve, or the slope of the chord connecting any two specified points on the stress-strain curve, may be considered as the modulus; in such cases the modulus is designated, respectively, as the "initial tangent modulus," the "tangent modulus," the "secant modulus," or the "chord modulus," and the stress stated. The modulus is expressed as force per unit of area (e.g., psi or Pa).

Modulus of rigidity—The ratio of unit shearing stress to the corresponding unit shearing strain; referred to as "shear modulus" and "modulus of elasticity in shear;" denoted by the symbol G. (See also **Modulus of elasticity.**)

Modulus of rupture—A measure of the ultimate load-carrying capacity of a beam and sometimes referred to as "rupture modulus" or "rupture strength." It is calculated for apparent tensile stress in the extreme fiber of a transverse test specimen under the load which produces rupture. (See also **Flexural strength.**)

Note: The actual stress in the extreme fiber is less than the apparent stress since the flexure formula employed in the calculation is valid only for stresses within the proportional limit of the material; nevertheless, the nominal rupture strength so obtained is considered the rupture modulus.

Modulus of subgrade reaction—Ratio of load per unit area of horizontal surface (of a mass of soil) to corresponding settlement of the surface; it is determined as the slope of the secant, drawn between the point corresponding to zero settlement and a specified point on the load-settlement curve obtained from a plate load test on a soil using a 30 in. or greater diameter loading plate.

Moist room—A room in which the atmosphere is maintained at a selected temperature (usually 23.0 ± 1.7 C or 73.4 ± 3.0 F) and a relative humidity of at least 98 percent, for the purpose of curing and storing cementitious test speciment; the facilities must be sufficient to maintain free moisture continuously on the exterior of test specimens.

Moisture movement—

1. The movement of moisture through a porous medium.

2. The effects of such movement on efflorescence and volume change in hardened cement paste, mortar, concrete, or rock.

See also **Shrinkage** and **Swelling.**

Mold—

1. A device containing a cavity into which neat cement, mortar, or concrete test specimens are cast.

2. A form used in the fabrication of precast mortar or concrete units (e.g., masonry units).

Mold oil—A mineral oil that is applied to the interior surface of a clean mold, before casting concrete or mortar therein, to facilitate removal of the mold after the concrete or mortar has hardened. (See also **Form oil, Bond breaker,** and **Release agent.**)

Moment—The colloquial expression for the more descriptive term bending moment. (See also **Bending moment.**)

Moment distribution—A method of structural analysis for continuous beams and rigid frames whereby successive converging corrections are made to an assumed set of moments until the desired precision is obtained; also known as the Hardy Cross method.

Monolith—A body of plain or reinforced concrete cast or erected as a single integral mass or structure.

Monolithic concrete—Concrete cast with no joints other than construction joints.

Monolithic surface treatment—See **Dry shake.**

Monolithic terrazzo—The application of a ⅝ in. (15 mm) terrazzo topping directly to a specially prepared concrete substrata, eliminating an underbed.

Monolithic topping—On flatwork: a higher quality, more serviceable topping course placed promptly after the base course has lost all slump and bleeding water.

Monomer—An organic liquid, of relatively low molecular weight, that creates a solid polymer by reacting with itself or other compounds of low molecular weight or both.

Monomolecular—Composed of single molecules; specifically, films that are one molecule thick. Denotes a thickness equal to one molecule (e.g., certain chemical compounds develop a "monomolecular film" over bleeding water at the surface of freshly placed concrete or mortar as a means of reducing the rate of evaporation). (See also **Evaporation retardant.**)

Montmorillonite—See **Montmorillonoid.**

Montmorillonoid—A group of clay minerals, including montmorillonite characterized by a sheet-like internal molecular structure; consisting of extremely finely-divided hydrous aluminum or magnesium silicates that swell on wetting, shrink on drying, and are subject to ion exchange.

Mortar—A mixture of cement paste and fine aggregate; in fresh concrete, the material occupying the interstices among particles of coarse aggregate; in masonry construction, mortar may contain masonry cement, or may contain hydraulic cement with lime (and possibly other admixtures) to afford greater plasticity and workability than are attainable with standard hydraulic cement mortar. (See also **Cement, masonry** and **Masonry**

Mosaic—Inlaid exposed surface designs of aggregates or other material.

Moving forms—Large prefabricated units of formwork incorporating supports, and designed to be moved horizontally on rollers or similar devices, with a minimum amount of dismantling between successive uses.

Mud sill—A timber or timber assembly bedded into the earth grade as a means of supporting framed construction.

Mud slab—A 2-in. to 6-in. (50-mm to 150-mm) layer of concrete below a structural concrete floor or footing over soft, wet soil; also called mud mat.

Multielement prestressing—Prestressing accomplished by stressing an assembly of several individual structural elements as a means of producing one integrated structural member.

Multistage stressing—Prestressing performed in stages as the construction progresses.

N

Nailable concrete—Concrete, usually made with a suitable lightweight aggregate, with or without the addition of sawdust, into which nails can be driven.

Nailer—A strip of wood or other fitting attached to or set in concrete, or attached to steel to facilitate making nailed connections.

Natural cement—See **Cement, natural.**

Natural sand—Sand resulting from natural disintegration and abrasion of rock. (See also **Sand** and **Fine aggregate.**)

Neat cement—Hydraulic cement in the unhydrated state.

Neat cement grout—A fluid mixture of hydraulic cement and water, with or without admixture; also the hardened equivalent of such mixture.

Neat cement paste—A mixture of hydraulic cement and water, both before and after setting and hardening.

Neat line—A line defining the proposed or specified limits of an excavation or structure.

Negative moment—A condition of flexure in which top fibers of a horizontally placed member, or external fibers of a vertically placed exterior member, are subjected to tensile stresses.

Negative reinforcement—Steel reinforcement for negative moment.

Net mixing water—See **Mixing water.**

Neutral axis—A line in the plane of a structural member subject to bending where the longitudinal stress is zero.

Nicol prism—A system of two optically clear crystals of calcite ("Iceland spar") used in producing plane-polarized light.

No-fines concrete—A concrete mixture containing little or no fine aggregate.

No-slump concrete—Concrete with a slump of ¼ in. (6 mm) or less. (See also **Zero-slump concrete.**)

Nominal maximum size of aggregate—In specifications for or descriptions of aggregate, the smallest sieve opening through which the entire amount of aggregate is *permitted* to pass. (See also **Maximum size of aggregate.**)

Note: Specifications on aggregates usually stipulates a sieve opening through which all the aggregate may, but not need, pass so that a stated maximum proportion of the aggregate may be retained on that sieve. A sieve opening so designated is the nominal size of the aggregate.

Non

Nominal mix—The proportions of the constituents of a proposed concrete mixture.

Nonagitating unit—A truck-mounted container, for transporting central-mixed concrete, not equipped to provide agitation (slow mixing) during delivery.

Non-air-entrained concrete—Concrete in which neither an air-entraining admixture nor air-entraining cement has been used.

Nonconcordant tendons—In statically indeterminate structures, tendons that are not coincident with the pressure line caused by the tendons. (See also **Cap cables.**)

Nonevaporable water—The water that is chemically combined during cement hydration; not removable by specified drying. (See also **Evaporable water.**)

Nonprestressed reinforcement—Reinforcing steel, not subjected to either pretensioning or post-tensioning.

Nonsimultaneous prestressing—The post-tensioning of tendons individually rather than simultaneously.

Normal consistency—

1. The degree of wetness exhibited by a freshly mixed concrete, mortar, or neat cement grout when the workability of the mixture is considered acceptable for the purpose at hand.

2. The physical condition of neat cement paste as determined with the Vicat apparatus in accordance with a standard method of test (e.g., ASTM C187).

Normal-weight concrete—See **Concrete, normal-weight.**

Nozzle—A metal or rubber tip attached to the discharge end of a heavy thick-wall rubber hose from which a continuous stream of shotcrete is ejected.

Nozzle liner—A replaceable rubber lining, fitted into the nozzle tip, to prevent abrasion of the interior surface of the nozzle.

Nozzle velocity—The rate at which shotcrete is ejected from the nozzle, usually stated in feet per second or meters per second.

Nozzleman—The operator who manipulates the nozzle and controls placement of the shotcrete; in the case of dry-mix shotcrete, the operator also controls the water content of the shotcrete.

O

Obsidian—A natural volcanic glass of relatively low water content. (See also **Perlite.**)

Offset—An abrupt change in alignment or dimension, either horizontally or vertically; a horizontal ledge occurring along a change in wall thickness of the wall above.

Offset bend—An intentional distortion from the normal straightness of a steel reinforcing bar in order to move the center line of a segment of the bar to a position parallel to the original position of the center line; a mechanical operation commonly applied to vertical bars that reinforce concrete columns.

Oil well cement—See **Cement, oil well.**

One-way system—The arrangement of steel reinforcement within a slab that presumably bends in only one direction.

Opal—A mineral composed of amorphous hydrous silica ($SiO_2 \cdot H_2O$).

Opaline chert—Chert composed entirely or mainly of opal.

Open-circuit grouting—A grouting system with no provision for recirculation of grout to the pump.

Open-top mixer—A truck-mounted mixer consisting of a trough or a segment of a cylindrical mixing compartment within which paddles or blades rotate about the horizontal axis of the trough. (See also **Mixer, horizontal shaft.**)

Orthotropic—A contraction of the terms "orthogonal anistropic" as in the phrase "orthogonal anistropic plate;" a hypothetical plate consisting of beams and a slab acting together with different flexural rigidities in the longitudinal and transverse directions, as in a composite beam bridge.

Ottawa sand—Silica sand produced by processing of material obtained by hydraulic mining of massive orthoquartzite situated in deposits near Ottawa, Illinois, composed almost entirely of naturally rounded grains of nearly pure quartz; used in mortars for testing of hydraulic cement. (See also **Standard sand** and **Graded standard sand.**)

Ovals—Marble chips which have been tumbled until a smooth oval shape has resulted.

Ovendry—The condition resulting from having been dried to essentially constant weight, in an oven, at a temperature which has been fixed, usually between 221 and 239 F (105 and 115 C).

Oven dry—To dry in an oven at a temperature usually between 221 and 239 F (105 and 115 C) until the weight of the test specimen becomes essentially constant.

Overdesign—To require adherence to structural design requirements higher than service demands, as a means of compensating for statistical variation or for anticipated deficiencies or both.

Overlay—A layer of concrete or mortar, seldom thinner than 1 in. (25 mm), placed on and usually bonded onto the worn or cracked surface of a concrete slab to either restore or improve the function of the previous surface.

Oversanded—Containing more sand than would be necessary to produce adequate workability and a satisfactory condition for finishing.

Overstretching—Stressing of tendons to a value higher than designed for the initial stress to: (a) overcome frictional losses, (b) temporarily overstress the steel to reduce steel creep that occurs after anchorage, and (c) counteract loss of prestressing force that is caused by subsequent prestressing of other tendons.

Overvibration—Excessive use of vibrators during placement of freshly mixed concrete, causing segregation and excessive bleeding.

P

Pack set—See **Sticky cement.**

Packaged concrete, mortar, grout—Mixtures of dry ingredients in packages, requiring only the addition of water to produce concrete, mortar, or grout.

Packer—A device inserted into a hole in which grout is to be injected which acts to prevent return of the grout around the injection pipe; usually an expandable device actuated mechanically, hydraulically, or pneumatically.

Packer-head process—A method of casting concrete pipe in a vertical position in which concrete of low water content is compacted with a revolving compaction tool.

Paddle mixer—See **Open-top mixer.**

Palladiana—See **Berliner.**

Pan—

1. A prefabricated form unit used in concrete joist floor construction.

2. A container that receives particles passing the finest sieve during mechanical analysis of granular materials.

Pan mixer—See **Mixer, pan.**

Panel—

1. A section of form sheathing, constructed from boards, plywood, metal sheets, etc., that can be erected and stripped as a unit.

2. A concrete member, usually precast, rectangular in shape, and relatively thin with respect to other dimensions.

Panel, drop—See **Drop panel.**

Panel strip—A strip extending across the length or width of a flat slab for structural design and construction or architectural purposes.

Paper form—A heavy paper mold used for casting concrete columns and other structural shapes.

Parallel-wire unit—A post-tensioning tendon composed of a number of wires or strands which are approximately parallel.

Parapet—That part of a wall that extends above the roof level; a low wall along the top of a dam.

Parge—To coat with plaster, particularly foundation walls and rough masonry.

Partial prestressing—Prestressing to a stress level such that, under design loads, tensile stresses exist in the precompressed tensile zone of the prestressed member.

Partial release—Release into a prestressed concrete member of a portion of the total prestress initially held wholly in the prestressed reinforcement.

Particle shape—The shape of a particle. (See also **Cubical piece, Elongated piece,** and **Flat piece.**)

Particle-size distribution—See **Grading.**

Parting agent—See **Release agent.**

Pass—Layer of shotcrete placed in one movement over the field of operation.

Paste content (of concrete)—Proportional volume of cement paste in concrete, mortar, or the like, expressed as volume percent of the entire mixture. (See also **Neat cement paste.**)

Paste volume (of concrete)—See **Paste content.**

Pat—A specimen of neat cement paste about 3 in. (76 mm) in diameter and ½ in. (13 mm) in thickness at the center and tapering to a thin edge on a flat glass plate for indicating setting time.

Pattern cracking—Fine openings on concrete surfaces in the form of a pattern; resulting from a decrease in volume of the material near the surface, or increase in volume of the material below the surface, or both.

Pavement (concrete)—A layer of concrete over such areas as roads, sidewalks, canals, playgrounds, and those used for storage or parking. (See also **Rigid pavement.**)

Paving train—An assemblage of equipment designed to place and finish a concrete pavement.

Pea gravel—Screened gravel, most of the particles of which will pass a ⅜ in. (9.5 mm) sieve and be retained on a No. 4 (4.75 mm) sieve.

Pedestal—An upright compression member whose height does not exceed three times its average least lateral dimension, such as a short pier or plinth used as the base for a column.

Pedestal pile—A cast-in-place concrete pile constructed so that concrete is forced out into a widened bulb or pedestal shape at the foot of the pipe which forms the pile.

Peeling—A process in which thin flakes of mortar are broken away from a concrete surface, such as by deterioration or by adherence of surface mortar to forms as forms are removed.

Pencil rod—Plain metal rod of about ¼ in. (6 mm) diameter.

Penetration probe—A device for obtaining a measure of the resistance of concrete to penetration; customarily determined by the distance that a steel pin is driven into the concrete from a special gun by a precisely measured explosive charge.

Penetration resistance—The resistance, usually expressed in pounds per square inch (psi) or megapascals (MPa), of mortar or cement paste to penetration by a plunger or needle under standard conditions.

Percent fines—Amount, expressed as a percentage, of material in aggregate finer than a given sieve, usually the No. 200 (75 μm) sieve; also the amount of fine aggregate in a concrete mixture expressed as a percent by absolute volume of the total amount of aggregate.

Percentage of reinforcement—The ratio of cross-sectional area of reinforcing steel to the effective cross-sectional area of a member, expressed as a percentage.

Periclase—A crystalline mineral, magnesia, MgO, the equivalent of which may be present in portland cement clinker, portland cement, and other materials such as open hearth slags, and certain basic refractories.

Perimeter grouting—Injection of grout, usually at relatively low pressure, around the periphery of an area which is subsequently to be grouted at greater pressure; intended to confine subsequent grout injection within the perimeter.

Period at maximum temperature—See **Maximum-temperature period.**

Perlite—A volcanic glass having a perlitic structure, usually having a higher water content than obsidian; when expanded by heating, used as an insulating material and as a lightweight aggregate in concretes, mortars, and plasters.

Perlitic structure—A structure produced in a homogeneous material by contraction during cooling, and consisting of a system of irregular convolute and spheroidal cracks; generally confined to natural glass.

Permanent form—Any form that remains in place after the concrete has developed its design strength; it may or may not become an integral part of the structure.

Permanent set—Inelastic elongation or shortening.

Permeability to water, coefficient of—The rate of discharge of water under laminar flow conditions through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature conditions usually 20 C.

Petrography—The branch of petrology dealing with description and systematic classification of rocks aside from their geologic relations, mainly by laboratory methods, largely chemical and microscopical; also, loosely, petrology or lithology.

Petrology—The science of rocks, treating of their origin, structure, composition, etc., from all aspects and in all relations. (See also **Petrography.**)

Phenolic resin—A class of synthetic, oil-soluble resins (plastics) produced as condensation products of phenol, substituted phenols and formaldehyde, or some similar aldehyde that may be used in paints for concrete.

Phi

Phi (ϕ) factor—Capacity reduction factor (in structural design); a number less than 1.0 (usually 0.65–0.90) by which the strength of a structural member or element (in terms of load, moment, shear, or stress) is required to be multiplied in order to determine design strength or capacity; the magnitude of the factor is stipulated in applicable codes and construction specifications for respective types of members and cross sections.

Philleo factor—A distance, used as an index of the extent to which hardened cement paste is protected from the effects of freezing, so selected that only a small portion of the cement paste (usually 10 percent) lies farther than that distance from the perimeter of the nearest air void. (See also **Protected paste volume.**)

Photometer—See **Flame photometer.**

Pier—Isolated foundation member of plain or reinforced concrete.

Pigment—A coloring matter, usually in the form of an insoluble fine powder.

Pilaster—Column built within a wall, usually projecting beyond the wall.

Pilaster face—The form for the front surface of a pilaster parallel to the wall.

Pilaster side—The form for the side surface of a pilaster perpendicular to the wall.

Pile—A slender timber, concrete, or steel structural element, driven, jetted, or otherwise embedded on end in the ground for the purpose of supporting a load or of compacting the soil. (See also **Composite pile.**)

Pile bent—Two or more piles driven in a row transverse to the long dimension of the structure and fastened together by capping and (sometimes) bracing.

Pile cap—

1. A structural member placed on, and usually fastened to, the top of a pile or a group of piles and used to transmit loads into the pile or group of piles and in the case of a group to connect them into a bent; also known as a rider cap or girder; also a masonry, timber, or concrete footing resting on a group of piles.

2. A metal cap or helmet temporarily fitted over the head of a precast pile to protect it during driving; some form of shock-absorbing material is often incorporated.

Pipe column—Column made of steel pipe; often filled with concrete.

Pipe pile—A steel cylinder, usually between 10 and 24 in. (250 and 600 mm) in diameter, generally driven with open ends to firm bearing and then excavated and filled with concrete; this pile may consist of several sections from 5 to 40 ft (1.5 to 8 m) long joined by special fittings such as cast-steel sleeves and is sometimes used with its lower end closed by a conical steel shoe.

Pitting—Development of relatively small cavities in a surface, due to phenomena such as corrosion or cavitation, or, in concrete, localized disintegration. (See also **Popout.**)

Placeability—See **Workability.**

Placement—The process of placing and consolidating concrete; a quantity of concrete placed and finished during a continuous operation; also inappropriately referred to as **Pouring.**

Placing—The deposition, distribution, and consolidation of freshly mixed concrete in the place where it is to harden; also inappropriately referred to as **Pouring.**

Plain bar—A reinforcing bar without surface deformations, or one having deformations that do not conform to the applicable requirements.

Plain concrete—Concrete without reinforcement; reinforced concrete that does not conform to the definition of reinforced concrete; also used loosely to designate concrete containing no admixture and prepared without special treatment.

Plane of weakness—The plane along which a body under stress will tend to fracture; may exist by design, by accident, or because of the nature of the structure and its loading.

Plaster—A cementitious material or combination of cementitious material and aggregate that, when mixed with a suitable amount of water, forms a plastic mass or paste which when applied to a surface, adheres to it and subsequently hardens, preserving in a rigid state the form or texture imposed during the period of plasticity; also the placed and hardened mixture. (See also **Stucco.**)

Plaster mold—A mold or form made from gypsum plaster, usually to permit concrete to be formed or cast in intricate shapes or in conspicuous relief. (See also **Mold** and **Form.**)

Plaster of Paris—$CaSO_4 \cdot \frac{1}{2}H_2O$, gypsum, from which three-quarters of the chemically bound water has been driven off by heating; when wetted it recombines with water and hardens quickly. (See also **Hemihydrate.**)

Plastic—Possessing plasticity, or possessing adequate plasticity. (See also **Plasticity.**)

Plastic centroid—Centroid of the resistance to load computed for the assumptions that the concrete is stressed uniformly to 0.85 its design strength and the steel is stressed uniformly to its specified yield point.

Plastic consistency—Condition of freshly mixed cement paste, mortar, or concrete such that deformation will be sustained continuously in any direction without rupture; in common usage, concrete with slump of 3 to 4 in. (80 to 100 mm).

Plastic cracking—Cracking that occurs in the surface of fresh concrete soon after it is placed and while it is still plastic.

Plastic deformation—Deformation that does not disappear when the force causing the deformation is removed.

Plastic design—See **Ultimate-strength design.**

Plastic flow—See **Creep.**

Plastic-hinge—Region where ultimate moment capacity in a member may be developed and maintained with corresponding significant inelastic rotation as main tensile steel elongates beyond yield strain.

Plastic limit—The water content at which a soil will just begin to crumble when rolled into a thread approximately ⅛ in. (3 mm) in diameter. (See also **Atterberg limits.**)

Plastic loss—See **Creep.**

Plastic mortar—A mortar of plastic consistency.

Plastic or bond fire clay—A fire clay of sufficient natural plasticity to bond nonplastic material; a fire clay used as a plasticizing agent in mortar.

Plastic shrinkage cracks—See **Plastic cracking.**

Plasticity—A complex property of a material involving a combination of qualities of mobility and magnitude of yield value; that property of freshly mixed cement paste concrete, or mortar which determines its resistance deformation or ease of molding.

Plasticity index—The range in water content through which a soil remains plastic; numerical difference between the liquid limit and the plastic limit. (See also **Atterberg limits.**)

Plasticizer—A material that increases plasticity of a cement paste, mortar, or concrete mixture.

Plasticizing—Producing plasticity or becoming plastic.

Plate—

1. In formwork for concrete: A flat, horizontal member at the top or bottom or both of studs or posts; a mudsill if on the ground.

2. In structural design: A member, the depth of which is substantially smaller than its length and width.

See also **Flat plate** and **Load-transfer assembly.**

Plum—A large random-shaped stone dropped into freshly placed mass concrete to economize on the volume of the concrete. (See also **Cyclopean concrete.**)

Plumb—Vertical or to make vertical.

Pneumatic feed—Shotcrete delivery equipment in which material is conveyed by a pressurized air stream.

Pneumatically applied mortar—See **Shotcrete.**

Point count—Method for determination of the volumetric composition of a solid by observation of the frequency with which areas of each component coincide with a regular system of points in one or more planes intersecting a sample of the solid.

Point count (modified)—The point count method supplemented by a determination of the frequency with which areas of each component of a solid are intersected by regularly spaced lines in one or more planes intersecting a sample of the solid.

Point load—A load whose area of contact with the resisting body is negligible in comparison with the area of the resisting body.

Point of contraflexure—See **Point of inflection.**

Point of inflection—The point on the length of a structural member subjected to flexure where the curvature changes from concave to convex or conversely and at which the bending moment is zero; also called "point of contraflexure"; location of an abrupt bend in a plotted locus of points in a graph.

Poisson's ratio—The ratio of transverse (lateral) strain to the corresponding axial (longitudinal) strain resulting from uniformly distributed axial stress below the proportional limit of the material; the value will average about 0.2 for concrete and 0.25 for most metals.

Polarizing microscope—A microscope equipped with elements permitting observations and determinations to be made using polarized light. (See also **Nicol prism.**)

Pole shore—See **Post shore.**

Polish or final grind—The final operation in which fine abrasives are used to hone a surface to its desired smoothness and appearance.

Polyester—One of a large group of synthetic resins, mainly produced by reaction of dibasic acids with dihydroxy alcohols; commonly prepared for application by mixing with a vinyl-group monomer and free-radical catalysts at ambient temperatures and used as binders for resin mortars and concretes, fiber laminates (mainly glass), adhesives, and the like. (See also **Polymer concrete.**)

Polyethylene—A thermoplastic high-molecular-weight organic compound used in formulating protective coatings or, in sheet form, as a protective cover for concrete surfaces during the curing period, or to provide a temporary enclosure for construction operations.

Polymer—The product of polymerization; more commonly a rubber or resin consisting of large molecules formed by polymerization.

Polymer concrete—Concrete in which an organic polymer serves as the binder (see **Concrete**); also known as resin concrete; sometimes erroneously employed to designate hydraulic cement mortars or concretes in which part or all of the mixing water is replaced by an aqueous dispersion of a thermoplastic copolymer.

Polymer-cement concrete—A mixture of water, hydraulic cement, aggregate, and a monomer or polymer; polymerized in place when a monomer is used.

Polymerization—The reaction in which two or more molecules of the same substance combine to form a compound containing the same elements, and in the same proportions, but of high molecular weight, from which the original substance can be generated, in some cases only with extreme difficulty.

Polystyrene resin—Synthetic resins varying in color from water-white to yellow formed by the polymerization of styrene on heating with or without catalysts that may be used in paints for concrete, or for making sculptured molds, or as insulation.

Polysulfide coating—A protective coating system prepared by polymerizing a chlorinated alkyl polyether with an inorganic polysulfide; exhibits outstanding resistance to ozone, sunlight, oxidation, and weathering.

Polyurethane—Reaction product of an isocyanate with any of a wide variety of other compounds containing an active hydrogen group; used to formulate tough, abrasion-resistant coatings.

Polyvinyl acetate—Colorless, permanently thermoplastic resin; usually supplied as an emulsion or water-dispersible powder characterized by flexibility, stability towards light, transparency to ultraviolet rays, high dielectric strength, toughness, and hardness; the higher the degree of polymerization, the higher the softening temperature; may be used in paints for concrete.

Polyvinyl chloride—A synthetic resin prepared by the polymerization of vinyl chloride, used in the manufacture of nonmetallic waterstops for concrete.

Pop-corn concrete—No-fines concrete containing insufficient cement paste to fill voids among the coarse aggregate so that the particles are bound only at points of contact. (See **No-fines concrete.**)

Popout—The breaking away of small portions of a concrete surface due to internal pressure which leaves a shallow, typically conical, depression.

Porosity—The ratio, usually expressed as a percentage, of the volume of voids in a material to the total volume of the material, including the voids.

Portland blast-furnace slag cement—See **Cement, portland blast-furnace slag.**

Portland cement—See **Cement, portland.**

Portland cement concrete—See **Concrete.**

Portland-pozzolan cement—See **Cement, portland-pozzolan.**

Portlandite—A mineral; calcium hydroxide ($Ca(OH)_2$); occurs naturally in Ireland; equivalent to a common product of hydration of portland cement.

Porous fill—See **Drainage fill.**

Positive displacement—Wet-mix shotcrete delivery equipment in which the material is pushed through the material hose in a solid mass by a piston or auger.

Pos

Positive moment.—A condition of flexure in which, for a horizontally simply supported member, the deflected shape is normally considered to be concave downward and the top fibers subjected to compression stresses; for other members and other conditions consider positive and negative as relative terms. (See also **Negative moment.**)

Note: For structural design and analysis, moments may be designated as positive or negative with satisfactory results as long as the sign convention adopted is used consistently.

Positive reinforcement—Reinforcement for positive moment.

Post—Vertical formwork member used as a brace; also shore, prop, jack.

Post shore or **Pole shore**—Individual vertical member used to support loads.

1. **Adjustable timber single-post shore**—Individual timber used with a fabricated clamp to obtain adjustment and not normally manufactured as a complete unit.

2. **Fabricated single-post shore**—Type I: Single all-metal post, with a fine-adjustment screw or device in combination with pin-and-hole adjustment or clamp; Type II: Single or double wooden post members adjustable by a metal clamp or screw and usually manufactured as a complete unit.

3. **Timber single-post shore**—Timber used as a structural member for shoring support.

Post-tensioning—A method of prestressing reinforced concrete in which tendons are tensioned after the concrete has hardened.

Pot life—Time interval after preparation during which a liquid or plastic mixture is usable.

Pouring (of concrete)—See **Placement** and **Placing.**

Power float—See **Rotary float.**

Powers spacing factor—See **Spacing factor.**

Pozzolan—A siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties.

Pozzolanic—Of or pertaining to a pozzolan.

Pozzolanic reaction—See **Pozzolan.**

Precast—A concrete member that is cast and cured in other than its final position; the process of placing and finishing precast concrete.

Precast concrete—Concrete cast elsewhere than its final position.

Precast pile—A reinforced concrete pile manufactured in a casting plant or at the site but not in its final position.

Precompressed zone—The area of a flexural member which is compressed by the prestressing tendons.

Precured period—See **Presteaming period.**

Prefiring—Raising the temperature of refractory concrete under controlled conditions prior to placing it in service.

Preformed foam—Foam produced in a foam generator prior to introduction of the foam into a mixer with other ingredients to produce cellular concrete.

Premature stiffening—See **False set** and **Flash set.**

Prepacked concrete—See **Concrete, preplaced-aggregate.**

Preplaced-aggregate concrete—See **Concrete, preplaced-aggregate** and **Colloidal concrete.**

Pre-post-tensioning—A method of fabricating prestressed concrete in which some of the tendons are pretensioned and a portion of the tendons are post-tensioned.

Preset period—See **Presteaming period.**

Preshrunk—

1. Concrete which has been mixed for a short period in a stationary mixer before being transferred to a transit mixer.

2. Grout, mortar, or concrete that has been mixed 1 to 3 hr before placing to reduce shrinkage during hardening.

Pressed edge—Edge of a footing along which the greatest soil pressure occurs under conditions of overturning.

Pressure line—Locus of force points within a structure resulting from combined prestressing force and externally applied load.

Presteaming period—In the manufacture of concrete products, the time between molding of a concrete product and start of the temperature-rise period.

Prestress—To place a hardened concrete member or an assembly of units in a state of compression prior to application of service loads; the stress developed by prestressing, such as by pretensioning or post-tensioning. (See also **Prestressed concrete, Prestressing steel, Pretensioning,** and **Post-tensioning.**)

Prestressed concrete—Concrete in which internal stresses of such magnitude and distribution are introduced that the tensile stresses resulting from the service loads are counteracted to a desired degree; in reinforced concrete the prestress is commonly introduced by tensioning the tendons.

Prestressing steel—High strength steel used to prestress concrete, commonly seven-wire strands, single wires, bars, rods, or groups of wires or strands. (See also **Prestress, Prestressed concrete, Pretensioning,** and **Post-tensioning.**)

Pretensioning—A method of prestressing reinforced concrete in which the tendons are tensioned before the concrete has hardened.

Pretensioning bed (or **Bench**)—The casting bed on which pretensioned members are manufactured and which resists the pretensioning force prior to release.

Primary crusher—A heavy crusher suitable for the first stage in a process of size reduction of rock, slag, or the like.

Primary nuclear vessel—Interior container in a nuclear reactor designed for sustained loads and for working conditions.

Principal planes—See **Principal stress.**

Principal stress—Maximum and minimum stresses at any point acting at right angles to the mutually perpendicular planes of zero shearing stress, which are designated as the **Principal planes.**

Probabilistic design—Method of design of structures using the principles of statistics (probability) as a basis for evaluation of structural safety.

Promoter—See **Catalyst.**

Proof stress—Stress applied to materials sufficient to produce a specified permanent strain; a specific stress to which some types of tendons are subjected in the manufacturing process as a means of reducing the deformation of anchorage, reducing the creep of steel, or insuring that the tendon is sufficiently strong.

Prop—See **Post** and **Shore.**

Proportional limit—The greatest stress which a material is capable of developing without any deviation from proportionality of stress to strain (Hooke's Law).

Proportioning—Selection of proportions of ingredients for mortar or concrete to make the most economical use of available materials to produce mortar or concrete of the required properties.

Protected corner—Corner of a slab with adequate provision for load transfer, such that at least 20 percent of the load from one slab corner to the corner of an adjacent slab is transferred by mechanical means or aggregate interlock.

Protected paste volume—The portion of hardened cement paste that is protected from the effects of freezing by proximity to an entrained air void. (See also **Philleo factor.**)

Proving ring—A device for calibrating load indicators of testing machines, consisting of a calibrated elastic ring and a mechanism or device for indicating the magnitude of deformation under load.

Psi—Abbreviation for "pounds per square inch."

Pulverized fuel ash (pfa)—See **Fly ash.**

Pumice—A highly porous and vesicular lava usually of relatively high silica content composed largely of glass drawn into approximately parallel or loosely entwined fibers, which themselves contain sealed vesicles.

Pumicite—Naturally occurring finely divided pumice.

Pumped concrete—Concrete which is transported through hose or pipe by means of a pump.

Pumping (of pavements)—The ejection of water, or water and solid materials such as clay or silt along transverse or longitudinal joints and cracks, and along pavement edges caused by downward slab movement activated by the passage of loads over the pavement after the accumulation of free water on or in the base course, subgrade, or subbase.

Punching shear—

1. Shear stress calculated by dividing the load on a column by the product of its perimeter and the thickness of the base or cap or by the product of the perimeter taken at one half the slab thickness away from the column and the thickness of the base or cap.

2. Failure of a base when a heavily loaded column punches a hole through it.

Punning—An obsolete term designating a light form of ramming. (See also **Ramming** and **Tamping.**)

Purlin—In roofs, a horizontal member supporting the common rafters.

Pycnometer—A vessel for determination of specific gravity of liquids or solids.

Pyrometric cone—A small, slender, three-sided oblique pyramid made of ceramic or refractory material for use in determining the time-temperature effect of heating and in obtaining the pyrometric cone equivalent (PCE) of refractory material.

Pyrometric cone equivalent (PCE)—The number of that cone whose tip would touch the supporting plaque simultaneously with that of a cone of the refractory material being investigated when tested in accordance with a specified procedure such as ASTM C 24.

Q

Quality assurance—A system of procedures for selecting the levels of quality required for a project or portion

thereof to perform the functions intended, and assuring that these levels are obtained.

Quality control—A system of procedures and standards by which a constructor, product manufacturer, materials processor, or the like, monitors the properties of the finished work.

Quartering—A method of obtaining a representative sample by dividing a circular pile of a larger sample into four equal parts and discarding opposite quarters successively until the desired size of sample is obtained.

Quicklime—Calcium oxide (CaO). (See also **Lime.**)

Quick set—See **Flash set** and **False set.**

R

Raft foundation—A continuous slab of concrete, usually reinforced, laid over soft ground or where heavy loads must be supported to form a foundation. (See also **Mat foundation.**)

Rail steel reinforcement—Reinforcing bars hot-rolled from standard T-section rails.

Rake classifier—Machine for separating coarse and fine particles of granular material temporarily suspended in water; the coarse particles settle to the bottom of a vessel and are scraped up an incline by a set of blades, the fine particles, remaining in suspension to be carried over the edge of the classifier.

Raker—A sloping brace for a shore head.

Raked joint—A joint in a masonry wall which has the mortar raked out to a specified depth while it is only slightly hardened.

Raking pile—See **Batter pile.**

Ramming—A form of heavy tamping of concrete, grout, or the like by means of a blunt tool forcibly applied. (See also **Dry pack, Punning,** and **Tamping.**)

Ranger—See **Wale.**

Raw mix—Blend of raw materials, ground to desired fineness, correctly proportioned, and blended ready for burning; such as that used in the manufacture of cement clinker.

Reactive aggregate—Aggregate containing substances capable of reacting chemically with the products of solution or hydration of the portland cement in concrete or mortar under ordinary conditions of exposure, resulting in some cases in harmful expansion, cracking, or staining.

Reactive silica material—Several types of materials which react at high temperatures with portland cement or lime during autoclaving; includes pulverized silica, natural pozzolan, and fly ash.

Ready-mixed concrete—Concrete manufactured for delivery to a purchaser in a plastic and unhardened state. (See **Central-mixed concrete, Shrink-mixed concrete,** and **Transit-mixed concrete.**)

Rebar—Abbreviation for "reinforcing bar." (See **Reinforcement.**)

Rebound—Aggregate and cement or wet shotcrete which bounces away from a surface against which shotcrete is being projected.

Rebound hammer—An apparatus that provides a rapid indication of the mechanical properties of concrete based on the distance of rebound of a spring-driven missile.

Refractories—Materials, usually nonmetallic, used to withstand high temperatures.

Ref

Refractoriness—In refractories, the property of being resistant to softening or deformation at high temperatures.

Refractory—Resistant to high temperatures.

Refractory aggregate—Materials having refractory properties which when bound together into a conglomerate mass by a matrix form a refractory body.

Refractory concrete—Concrete having refractory properties, and suitable for use at high temperatures (generally about 315 to 1315 C), in which the binding agent is a hydraulic cement.

Refractory insulating concrete—Refractory concrete having low thermal conductivity.

Reglet—A groove in a wall to receive flashing.

Reinforced concrete—Concrete containing adequate reinforcement (prestressed or not prestressed) and designed on the assumption that the two materials act together in resisting forces. (See also **Plain concrete.**)

Reinforced masonry—Unit masonry in which reinforcement is embedded in such a manner that the two materials act together in resisting forces.

Reinforcement—Bars, wires, strands, and other slender members which are embedded in concrete in such a manner that the reinforcement and the concrete act together in resisting forces.

Reinforcement, cold-drawn wire—Steel wire made from rods that have been hot rolled from billets, cold-drawn through a die; for concrete reinforcement of small diameter such as in gages not less than 0.080 in. (2 mm) nor greater than 0.625 in. (16 mm).

Reinforcement, cold-worked steel—Steel bars or wires which have been rolled, twisted, or drawn at normal ambient temperatures.

Reinforcement, distribution-bar—Small-diameter bars, usually at right angles to the main reinforcement, intended to spread a concentrated load on a slab and to prevent cracking.

Reinforcement, dowel-bar—See **Dowel.**

Reinforcement, expanded metal fabric—See **Expanded metal lath.**

Reinforcement, four-way—A system of reinforcement in flat slab construction comprising bands of bars parallel to two adjacent edges and also to both diagonals of a rectangular slab.

Reinforcement, helical—Steel reinforcement of hot rolled bar or cold drawn wire fabricated into a helix (more commonly known as spiral reinforcement).

Reinforcement, high strength—Concrete reinforcing bars having a minimum yield of 60,000 psi (414 MPa).

Reinforcement, hoop—A one-piece closed tie or continuously wound tie not less than #3 in size, the ends of which have a standard 135-deg bend with a ten-bar-diameter extension, that encloses the longitudinal reinforcement.

Reinforcement, lateral—Usually applied to ties, hoops, and spirals in columns or column-like members.

Reinforcement, mesh—See **Welded-wire fabric** and **Welded-wire fabric reinforcement.**

Reinforcement, principal—Elements or configurations of reinforcement that provide the main resistance of reinforced concrete to loads borne by structures. (See also **Reinforcement, secondary.**)

Reinforcement, secondary—Reinforcement other than main reinforcement.

Reinforcement, spiral—See **Spiral reinforcement.**

Reinforcement, transverse—Reinforcement at right angles to the longitudinal reinforcement; may be main or secondary reinforcement.

Reinforcement, twin-twisted bar—Two bars of the same nominal diameter twisted together.

Reinforcement, two-way—Reinforcement arranged in bands of bars at right angles to each other.

Reinforcement, welded—Reinforcement joined together by welding.

Reinforcement displacement—Movement of reinforcing steel from its specified position in the forms.

Reinforcement ratio—Ratio of the effective area of the reinforcement to the effective area of the concrete at any section of a structural member. (See also **Percentage of reinforcement.**)

Relative humidity—The ratio of the quantity of water vapor actually present to amount present in a saturated atmosphere at a given temperature; expressed as a percentage.

Relaxation (of steel)—Decrease in stress in steel as a result of creep within the steel under prolonged strain; decrease in stress in steel as a result of decreased strain of the steel, such as results from shrinkage and creep of the concrete in a prestressed concrete unit.

Release agent—Material used to prevent bonding of concrete to a surface. (See also **Bond breaker.**)

Remoldability—The readiness with which freshly mixed concrete responds to a remolding effort such as jigging or vibration causing it to reshape its mass around reinforcement and to conform to the shape of the form. (See also **Flow.**)

Remolding test—A test to measure remoldability.

Rendering—The application, by means of a trowel or float, of a coat of mortar.

Repeatability—Variability among replicate test results obtained on the same material within a single laboratory by one operator; a quantity that will be exceeded in only about 5 percent of the repetitions by the difference, taken in absolute value, of two randomly selected test results obtained in the same laboratory on a given material; in use of the term all variable factors should be specified.

Reposting—See **Reshoring.**

Reproducibility—Variability among replicate test results obtained on the same material in different laboratories; a quantity that will be exceeded in only about 5 percent of the repetitions by the difference, taken in absolute value, of two single test results made on the same material in two different, randomly selected laboratories; in use of the term all variable factors should be specified.

Resetting (of forms)—Setting of forms separately for each successive lift of a wall to avoid offsets at construction joints.

Reshoring—The construction operation in which the original shoring or posting is removed and replaced in such a manner as to avoid deflection of the shored element or damage to partially cured concrete.

Resilience—The work done per unit volume of a material in producing strain.

Resin—A natural or synthetic, solid or semisolid organic material of indefinite and often high molecular weight having a tendency to flow under stress, usually has a softening or melting range and usually fractures conchoidally.

Resin mortar (or **Concrete**)—See **Polymer concrete.**

Restraint (of concrete)—Restriction of free movement of fresh or hardened concrete following completion of placing in formwork or molds or within an otherwise confined space; restraint can be internal or external and may act in one or more directions.

Retardation—Reduction in the rate of hardening or setting, i.e., an increase in the time required to reach initial and final set or to develop early strength of fresh concrete, mortar, or grout. (See also **Retarder.**)

Retarder—An admixture which delays the setting of cement paste, and hence of mixtures such as mortar or concrete containing cement.

Retempering—Addition of water and remixing of concrete or mortar which has lost enough workability to become unplaceable or unusable. (See also **Tempering.**)

Reveal—The side of an opening in a wall for a window or door; depth of exposure of aggregate in an exposed aggregate finish. (See also **Exposed-aggregate finish.**)

Revibration—One or more applications of vibration to concrete after completion of placing and initial compaction but preceding initial setting of the concrete.

Revolving-blade (or **paddle**) **mixer**—See **Open-top mixer.**

Rheology—The science dealing with flow of materials, including studies of deformation of hardened concrete, the handling and placing of freshly mixed concrete, and the behavior of slurries, pastes, and the like.

Rib—One of a number of parallel structural members backing sheathing; the portion of a T-beam which projects below the slab; in deformed reinforcing bars, the deformations or the longitudinal parting ridge.

Ribbed panel—A panel composed of a thin slab reinforced by a system of ribs in one or two directions, usually orthogonal.

Ribbed slab—See **Ribbed panel.**

Ribbon—A narrow strip of wood or other material used in formwork.

Ribbon loading—Method of batching concrete in which the solid ingredients, and sometimes also the water, enter the mixer simultaneously.

Rich concrete—Concrete of high cement content. (See also **Lean concrete.**)

Rich mixture—A concrete mixture containing a high proportion of cement.

Rider cap—See **Pile cap.**

Rigid frame—A frame depending on moment in joints for stability.

Rigid pavement—Pavement that will provide high bending resistance and distribute loads to the foundation over a comparatively large area.

Rock pocket—A porous, mortar-deficient portion of hardened concrete consisting primarily of coarse aggregate and open voids, caused by leakage of mortar from form, separation (segregation) during placement, or insufficient consolidation. (See also **Honeycomb.**)

Rod—Sharp-edged cutting screed used to trim shotcrete to forms or ground wires. (See also **Screed.**)

Rod, Dowel—See **Dowel.**

Rod, tamping—A round, straight, steel rod having one or both ends rounded to a hemispherical tip.

Rodability—The susceptibility of fresh concrete or mortar to compaction by means of a tamping rod.

Rod buster (colloquial)—One who installs reinforcement for concrete.

Rodding—Compaction of concrete or the like by means of a tamping rod. (See also **Rod, tamping** and **Rodability.**)

Rolling—The use of heavy metal or stone rollers on terrazzo topping to extract excess matrix.

Roman cement—A misnomer for a hydraulic cement made by calcining a natural mixture of calcium carbonate and clay, such as argillaceous limestone, to a temperature below that required to sinter the material but high enough to decarbonate the calcium carbonate, followed by grinding, so named because its brownish color resembles ancient Roman cements produced by use of lime-pozzolan mixtures.

Roof insulation—Lightweight concrete used primarily as insulating material over structural roof systems.

Rosiwal method—See **Linear traverse method.**

Rotary float (also called **Power float**)—A motor-driven revolving disc that smooths, flattens, and compacts the surface of concrete floors or floor toppings.

Rotary kiln—A long steel cylinder with a refractory lining, supported on rollers so that it can rotate about its own axis, and erected with a slight inclination from the horizontal so that prepared raw materials fed into the higher end move to the lower end, where fuel is blown in by air blast. (See also **Kiln, cement.**)

Rough grind—The initial operation in which coarse abrasives are used to cut the projecting chips in hardened terrazzo down to a level surface.

Rout—To deepen and widen a crack to prepare it for patching or sealing.

Rub brick—See **Rubbing brick.**

Rubbing brick—A silicon-carbide brick used to smooth and remove irregularities from surfaces of hardened concrete.

Rubbed finish—A finish obtained by using an abrasive to remove surface irregularities from concrete. (See also **Sack rub.**)

Rubber set—See **False set.**

Rubble—Rough stones of irregular shape and size, broken from larger masses by geological processes or by quarrying.

Rubble concrete—

1. Concrete similar to cyclopean concrete except that small stones (such as one man can handle) are used.
2. Concrete made with rubble from demolished structures.

See also **Cyclopean concrete.**

Runway—Decking over area of concrete placement, usually of movable panels and supports, on which buggies of concrete travel to points of placement.

Rupture modulus—See **Modulus of rupture.**

Rupture strength—See **Modulus of rupture.**

Rustic or washed finish—A type of terrazzo topping in which the matrix is recessed by washing prior to setting so as to expose the chips without destroying the bond between chip and matrix; a retarder is sometimes applied to the surface to facilitate this operation. (See also **Exposed aggregate finish.**)

Rustication—A groove in a concrete or masonry surface.

Rustication strip—A strip of wood or other material attached to a form surface to produce a groove or rustication in the concrete.

S

Sack—See **Bag.**

Sack rub—A finish for formed concrete surfaces, designed to produce even texture and fill all pits and air holes (see **Bug holes**); after dampening the surface, mortar is rubbed over surface; then, before it dries, a mixture of dry cement and sand is rubbed over it with a wad of burlap or a sponge-rubber float to remove surplus mortar and fill voids.

Safe leg load—The load which can safely be directly imposed on the frame leg of a scaffold. (See also **Allowable load.**)

Sagging—Subsidence of shotcrete material from a sloping, vertical, or overhead placement; also, the condition of a horizontal structural member bending downward under load. (See also **Sloughing.**)

Salamander—A portable source of heat, customarily oil-burning, used to heat an enclosure around or over newly placed concrete to prevent the concrete from freezing.

Sand—

1. Granular material passing the ⅜-in. (9.5-mm) sieve and almost entirely passing the No. 4 (4.75-mm) sieve and predominantly retained on the No. 200 (75-μm) sieve, and resulting from natural disintegration and abrasion of rock or processing of completely friable sandstone; or

2. that portion of an aggregate passing the No. 4 (4.75-mm) sieve and predominantly retained on the No. 200 (75-μm) sieve, and resulting from natural disintegration and abrasion of rock or processing of completely friable sandstone.

See also **Fine aggregate.**

Note: The definitions are alternatives to be applied under differing circumstances. Definition (1) is applied to an entire aggregate either in a natural condition or after processing. Definition (2) is applied to a portion of an aggregate. Requirements for properties and grading should be stated in the specifications. Fine aggregate produced by crushing rock, gravel, or slag commonly is known as **Manufactured sand.**

Sandblast—A system of cutting or abrading a surface such as concrete by a stream of sand ejected from a nozzle at high speed by compressed air; often used for cleanup of horizontal construction joints or for exposure of aggregate in architectural concrete.

Sand box (or **Sand jack**)—A tight box filled with clean, dry, sand on which rests a tight-fitting timber plunger that supports the bottom of posts used in centering; removal of a plug from a hole near the bottom of the box permits the sand to run out when it is necessary to lower the centering.

Sand-coarse aggregate ratio—Ratio of fine to coarse aggregate in a batch of concrete, by weight or volume.

Sand equivalent—A measure of the relative proportions of detrimental fine dust or claylike material in soils or fine aggregate.

Sand grout—Any grout in which fine aggregate is incorporated into the mixture; also termed sanded grout.

Sanded grout—See **Sand grout.**

Sand jack—See **Sand box.**

Sand-lime brick—See **Calcium-silicate brick.**

Sand, Ottawa—See **Ottawa sand.**

Sand plate—A flat steel plate or strip welded to the legs of bar supports for use on compacted soil.

Sand pocket—A zone in concrete or mortar containing sand without cement.

Sandstone—A cemented or otherwise compacted sedimentary rock composed predominantly of sand grains.

Sand streak—A streak of exposed fine aggregate in the surface of formed concrete caused by bleeding.

Sandwich panel—A prefabricated panel which is a layered composite, formed by attaching two thin facings to a thicker core; such as a precast concrete panel consisting of two layers of concrete separated by a nonstructural insulating core.

Santorin earth—A volcanic tuff originating on the Grecian island of Santorin and used as a pozzolan.

Saturated surface dry—Condition of an aggregate particle or other porous solid when the permeable voids are filled with water and no water is on the exposed surfaces.

Saturation—Act or process of saturating, or state of being saturated.

Saw cut—A cut in hardened concrete utilizing diamond or silicone-carbide blades or discs.

Sawdust concrete—Concrete in which the aggregate consists mainly of sawdust from wood.

Sawed joint—A joint cut in hardened concrete, generally not to the full depth of the member, by means of special equipment.

Scab—A short piece of wood fastened to two formwork members to secure a butt joint.

Scaffolding—A temporary structure for the support of deck forms, cartways, or workmen, or a combination of these such as an elevated platform for supporting workmen, tools, and materials; adjustable metal scaffolding is frequently adapted for shoring in concrete work.

Scaling—Local flaking or peeling away of the near-surface portion of hardened concrete or mortar; also of a layer from metal. (See also **Peeling, Spalling,** and **Mill scale.**)

Note: **Light scaling** of concrete does not expose coarse aggregate; **medium scaling** involves loss of surface mortar to 5 to 10 mm in depth and exposure of coarse aggregate; **severe scaling** involves loss of surface mortar to 5 to 10 mm in depth with some loss of mortar surrounding aggregate particles 10 to 20 mm in depth; **very severe scaling** involves loss of coarse aggregate particles as well as mortar generally to a depth greater than 20 mm.

Scalper—A sieve for removing oversize particles.

Scalping—The removal of particles larger than a specified size by sieving.

Scarf connection—A connection made by precasting, beveling, halving, or notching two pieces to fit together; after overlapping, the pieces are secured by bolts or other means.

Scarf joint—See **Scarf connection.**

Scoria—Vesicular volcanic ejecta of larger size, usually of basic composition and characterized by dark color; the material is relatively heavy and partly glassy, partly crystalline; the vesicles do not generally interconnect. (See also **Aggregate, lightweight.**)

Scour—Erosion of a concrete surface, exposing the aggregate.

Scratch coat—The first coat of plaster or stucco applied to a surface in three-coat work; usually cross-raked or scratched to form a mechanical key with the brown coat.

Screed—

1. To strike off concrete lying above the desired plane or shape.

2. A tool for striking off the concrete surface, sometimes referred to as a **Strikeoff.**

Screed guide—Firmly established grade strips or side forms for unformed concrete which will guide the strikeoff in producing the desired plane or shape.

Screed wire—See **Ground wire.**

Screeding—The operation of forming a surface by the use of screed guides and a strikeoff. (See also **Strikeoff.**)

Screen—See **Sieve.**

Screen analysis—See **Sieve analysis.**

Sealant—See **Joint sealant** and **Membrane curing.**

Sealing compound—See **Joint sealant** and **Membrane curing.**

Secant modulus—See **Modulus of elasticity.**

Secondary beam—A flexural member that is not a portion of the principal structural frame of a structure.

Secondary crusher—A crusher used for the second stage in a process of size reduction. (See also **Primary crusher.**)

Secondary moment—In statically indeterminate structures, the additional moments caused by deformation of the structure due to the applied forces; in statically indeterminate prestressed concrete structures, the additional moments caused by the use of a nonconcordant prestressing tendon.

Secondary nuclear vessel—Exterior container or safety container in a nuclear reactor subjected to design load only once in its lifetime, if at all.

Section modulus—A term pertaining to the cross section of a flexural member; the section modulus with respect to either principal axis is the moment of inertia with respect to either principal axis is the moment of inertia with respect to that axis divided by the distance from that axis to the most remote point of the tension or compression area of the section, as required; the section modulus is used to determine the flexural stress in a beam.

Segmental member—A structural member made up of individual elements prestressed together to act as a monolithic unit under service loads.

Segregation—The differential concentration of the components of mixed concrete, aggregate, or the like, resulting in nonuniform proportions in the mass. (See also **Separation.**)

Self-desiccation—The removal of free water by chemical reaction so as to leave insufficient water to cover the solid surfaces and to cause a decrease in the relative humidity of the system; applied to an effect occurring in sealed concretes, mortars, and pastes.

Self-furring—Metal lath or welded wire fabric formed in the manufacturing process to include means by which the material is held away from the supporting surface, thus creating a space for "keying" of the insulating concrete, plaster, or stucco.

Self-furring nail—Nails with flat heads and a washer or a spacer on the shank; for fastening reinforcing wire mesh and spacing it from the nailing member.

Self-stressing concrete (mortar or grout)—Expansive-cement concrete (mortar or grout) in which expansion, if restrained, induces persistent compressive stresses in the concrete (mortar or grout); also known as chemically prestressed concrete.

Semiautomatic batcher—See **Batcher.**

Semiflexible joint—A connection in which the reinforcement is arranged to permit some rotation of the joint.

Separation—The tendency, as concrete is caused to pass from the unconfined ends of chutes or conveyor belts, or similar arrangements, for coarse aggregate to separate from the concrete and accumulate at one side; the tendency, as processed aggregate leaves the ends of conveyor belts, chutes, or similar devices with confining sides, for the larger aggregate to separate from the mass and accumulate at one side; or the tendency for the solids to separate from the water by gravitational settlement. (See also **Bleeding** and **Segregation.**)

Sequence-stressing loss—In post-tensioning, the elastic loss in a stressed tendon resulting from the shortening of the member when additional tendons are stressed.

Service dead load—The dead weight supported by a member.

Service live load—The live load specified by the general building code or bridge specification, or the actual nonpermanent load applied in service.

Service load—See **Service dead load** and **Service live load.**

Set—The condition reached by a cement paste, mortar, or concrete when it has lost plasticity to an arbitrary degree, usually measured in terms of resistance to penetration or deformation; initial set refers to first stiffening; final set refers to attainment of significant rigidity; also, strain remaining after removal of stress. (See **Permanent set.**)

Setting shrinkage—A reduction in volume of concrete prior to the final set of cement, caused by settling of the solids and by the decrease in volume due to the chemical combination of water with cement.

Setting time—See **Initial setting time** and **Final setting time.**

Settlement—Sinking of solid particles in grout, mortar, or fresh concrete, after placement and before initial set. (See also **Bleeding.**)

Settling—The lowering in elevation of sections of pavement or structures due to their mass, the loads imposed on them, or shrinkage or displacement of the support.

Shale—A laminated and fissile sedimentary rock, the constituent particles of which are principally in clay and silt sizes; the laminations bedding planes of rock.

Sharp sand—Coarse sand of which the particles are of angular shape.

She bolt—A type of form tie and spreader bolt in which the end fastenings are threaded into the end of the bolt, thus eliminating cones and reducing the size of holes left in the concrete surface.

Shear—An internal force tangential to the plane on which it acts. (See also **Shearing force.**)

Shearhead—Assembled unit in the top of the columns of flat slab or flat plate construction to transmit loads from slab to column.

Shear modulus—See **Modulus of rigidity.**

Shear reinforcement—Reinforcement designed to resist shear or diagonal tension stresses. (See **Dowel.**)

Shear strength—The maximum shearing stress which a material or structural member is capable of developing, based on the original area of cross section.

Shearwall—A wall portion of a structural frame intended to resist lateral forces, such as earthquake, wind, and blast, acting in or parallel to the plane of the wall.

Shearing force—The algebraic sum of all the tangential forces acting on either side of the section at a particular location in a flexural member.

She

Sheath—An enclosure in which post-tensioning tendons are encased to prevent bonding during concrete placement. (See **Duct.**)

Sheathing—The material forming the contact face of forms; also called lagging or sheeting.

Sheet pile—A pile in the form of a plank driven in close contact or interlocking with others to provide a tight wall to resist the lateral pressure of water, adjacent earth, or other materials; may be tongued and grooved if made of timber or concrete and interlocking if made of metal.

Sheeting—See **Sheathing.**

Shelf angles—Structural angles with holes or slots in one leg for bolting to the structure to support brick work, stone, or terra cotta.

Shelf life—Maximum interval during which a material may be stored and remain in a usable condition.

Shell construction—Construction using thin curved slabs.

Shelly structure—See **Perlitic structure.**

Shielding concrete—Concrete, employed as a biological shield to attenuate or absorb nuclear radiation, usually characterized by high specific gravity or high hydrogen (water) content or boron content, having specific radiation attenuation effects. (See also **Biological shielding.**)

Shim—A strip of metal, wood, or other material employed to set base plates or structural members at the proper level for placement of grout, or to maintain the elongation in some types of post-tensioning anchorages.

Shiplap—A type of joint in lumber or precast concrete, made by using pieces having a portion of the width cut away on both edges, but on opposite sides, so as to make a flush joint with similar pieces.

Shock—See **Thermal shock.**

Shock load—Impact of material such as aggregate or concrete as it is released or dumpted during placement.

Shooting—Placing of shotcrete. (See also **Gunning.**)

Shore—A temporary support for formwork and fresh concrete or for recently built structures which have not developed full design strength; also called **Prop, Tom, Post, strut.** (See also **L-head and T-head.**)

Shore head—Wood or metal horizontal member placed on and fastened to vertical shoring member. (See also **Raker.**)

Shoring—Props or posts of timber or other material in compression used for the temporary support of excavations, formwork, or unsafe structures; the process of erecting shores.

Shoring, horizontal—Metal or wood load-carrying strut, beam, or trussed section used to carry a shoring load from one bearing point, column, frame, post, or wall to another; may be adjustable.

Shoring layout—A drawing prepared prior to erection showing arrangement of equipment for shoring.

Short column—See **Column, short.**

Shotcrete—Mortar or concrete pneumatically projected at high velocity onto a surface; also known as air-blown mortar; also pneumatically applied mortar or concrete, sprayed mortar and gunned concrete. (See also **Dry-mix shotcrete, Pneumatic feed, Positive displacement,** and **Wet-mix shotcrete.**)

Shoulder—An unintentional offset in a formed concrete surface usually caused by bulging or movement of formwork.

Shrink-mixed concrete—Ready-mixed concrete mixed partially in a stationary mixer and then mixed in a truck mixer. (See also **Preshrunk.**)

Shrinkage—Volume decrease caused by drying and chemical changes; a function of time but not of temperature or of stress due to external load. (See also **Contraction** and **Expansion.**)

Shrinkage, drying—See **Drying shrinkage** and **Shrinkage.**

Shrinkage-compensating—A characteristic of grout, mortar, or concrete made using an expansive cement in which volume increase if restrained, induces compressive stresses which are intended to approximately offset the tendency of drying shrinkage to induce tensile stresses. (See also **Expansive cement.**)

Shrinkage crack—Crack due to restraint of shrinkage.

Shrinkage cracking—Cracking of a structure or member due to failure in tension caused by external or internal restraints as reduction in moisture content develops, or as carbonation occurs, or both.

Shrinkage limit—The maximum water content at which a reduction in water content will not cause a decrease in volume of the soil mass. (See also **Atterberg limits.**)

Shrinkage loss—Reduction of stress in prestressing steel resulting from shrinkage of concrete.

Shrinkage reinforcement—Reinforcement designed to resist shrinkage stresses in concrete.

Shuttering—See **Formwork.**

Sieve—A metallic plate or sheet, a woven wire cloth, or other similar device, with regularly spaced apertures of uniform size, mounted in a suitable frame or holder for use in separating material according to size; in mechanical analysis, an apparatus with square openings is a sieve, one with circular apertures is a screen.

Sieve analysis—Determination of the proportions of particles lying within certain size ranges in a granular material by separation on sieves of different size openings. (See also **Grading.**)

Sieve correction—Correction of a sieve analysis to adjust for deviation of sieve performance from that of standard calibrated sieves.

Sieve number—A number used to designate the size of a sieve, usually the approximate number of openings per linear inch; applied to sieves with openings smaller than 6.3 mm (¼ in.).

Sieve size—Nominal size of openings between cross wires of a testing sieve, usually.

Silica—Silicon dioxide (SiO_2).

Silica flour—Very finely divided silica, a siliceous binder component which reacts with lime under autoclave curing conditions; prepared by grinding silica, such as quartz, to a fine powder; also known as silica powder.

Silica powder—See **Silica flour.**

Silicate—Salt of a silicic acid.

Silicon carbide—An artificial product (SiC) granules of which may be embedded in concrete surfaces to increase resistance to wear or as a means of reducing skidding or slipping on stair treads or pavements; also used as an abrasive in saws and drills for cutting concrete and masonry.

Silicone—A resin, characterized by water-repellant properties, in which the main polymer chain consists of alternating silicon and oxygen atoms, with carbon-containing side groups; silicones may be used in caulking or coating compounds or admixtures for concrete.

Sill—See **Mud sill.**

Silt—A granular material resulting from the disintegration of rock, with grains largely passing a No. 200 (75 μm) sieve; alternatively, such particles in the range from 2 to 50 μm diameter.

Simple beam—A beam without restraint or continuity at its supports.

Single-sized aggregate—Aggregate in which a major portion of the particles are of sizes lying between narrow limits.

Single-stage curing—Autoclave curing process in which precast concrete products are put on metal pallets for autoclaving and remain there until stacked for delivery or yard storage.

Sintering—The formation of a porous mass of material by the agglomeration of fine particles by partial fusion.

Sintering grate—A grate on which material is sintered.

Skew back—Sloping surface against which the end of an arch rests, such as a concrete thrust block supporting thrust of an arch bridge. (See also **Chamfer strip.**)

Skid resistance—A measure of the frictional characteristics of a surface.

Slab—A flat, horizontal or nearly so, molded layer of plain or reinforced concrete, usually of uniform but sometimes of variable thickness, either on the ground or supported by beams, columns, walls, or other framework. (See also **Flat slab** and **Flat plate.**)

Slab bolster—See **Bolster, slab.**

Slab spacer—Bar support and spacer for slab reinforcement, similar to slab bolster but without corrugations in top wire, no longer in general use. (See also **Bolster, slab.**)

Slab strip—See **Middle strip.**

Slag—See **Blast-furnace slag.**

Slag cement—See **Cement, slag.**

Slate—A fine-grained metamorphic rock possessing a well-developed fissility (slaty cleavage) usually not parallel to the bedding planes of the rock.

Slender beam—A beam, which if loaded to failure without lateral bracing of the compression flange, would fail by buckling rather than in flexure.

Slenderness ratio—The ratio of effective length or height of a wall, column, or pier to the radius of gyration; used as a means of assessing the stability of the element.

Slick line—End section of a pipe line used in placing concrete by pump which is immersed in the placed concrete and moved as the work progresses.

Sliding form—See **Slipform.**

Slip—Movement occurring between steel reinforcement and concrete in stressed reinforced concrete indicating anchorage breakdown.

Slipform—A form which is pulled or raised as concrete is placed; may move in a generally horizontal direction to lay concrete evenly for highway paving or on slopes and inverts of canals, tunnels, and siphons; or vertically to form walls, bins, or silos.

Sloped footing—A footing having sloping top or side faces.

Sloughing—Subsidence of shotcrete, due generally to excessive water in mixture; also called sagging. (See also **Sagging.**)

Slugging—Pulsating and intermittent flow of shotcrete material due to improper use of delivery equipment and materials.

Slump—A measure of consistency of freshly mixed concrete, mortar, or stucco equal to the subsidence measured to the nearest ¼ in. (6 mm) of the molded specimen immediately after removal of the slump cone.

Slump cone—A mold in the form of the lateral surface of the frustum of a cone with a base diameter of 8 in. (203 mm), top diameter 4 in. (102 mm), and height 12 in. (305 mm), used to fabricate a specimen of freshly mixed concrete for the slump test; a cone 6 in. (152 mm) high is used for tests of freshly mixed mortar and stucco.

Slump loss—The amount by which the slump of freshly mixed concrete changes during a period of time after an initial slump test was made on a sample or samples thereof.

Slump test—The procedure for measuring slump.

Slurry—A mixture of water and any finely divided insoluble material, such as portland cement, slag, or clay in suspension.

Slush grouting—Distribution of a grout with or without fine aggregate as required over a rock or concrete surface which is subsequently to be covered with concrete, usually by brooming it into place to fill surface voids and fissures.

Snap tie—A proprietary concrete wall-form tie, the end of which can be twisted or snapped off after the forms have been removed.

Soaking period—In high-pressure and low-pressure steam curing, the time during which the live steam supply to the kiln or autoclave is shut off and the concrete products are exposed to the residual heat and moisture.

Soffit—The underside of a part or member of a structure, such as a beam, stairway, or arch.

Soft particle—An aggregate particle possessing less than an established degree of hardness or strength as determined by a specific testing procedure.

Soil—A generic term for unconsolidated natural surface material above bedrock.

Soil pressure—See **Contact pressure.**

Soldier—A vertical wale used to strengthen or align formwork or excavations.

Solid panel—A solid slab, usually of constant thickness.

Solid volume—See **Absolute volume.**

Solution—A liquid consisting of at least two substances, one of which is a liquid solvent in which the other or others, which may be solid or liquid, are dissolved.

Solvent—A liquid in which another substance may be dissolved.

Sonic modulus—See **Dynamic modulus of elasticity.**

Sounding well—A vertical conduit in the mass of coarse aggregate for preplaced aggregate concrete, provided with continuous or closely spaced openings to permit entrance of grout; the grout level is determined by means of a float on a measured line.

Soundness—The freedom of a solid from cracks, flaws, fissures, or variations from an accepted standard; in the case of a cement, freedom from excessive volume change after setting; in the case of aggregate, the ability to withstand the aggressive action to which concrete containing it might be exposed, particularly that due to weather.

Spacer—Device which maintains reinforcement in proper position, or wall forms at a given distance apart before and during concreting. (See also **Spreader.**)

Spacing factor—An index related to the maximum distance of any point in a cement paste or in the cement paste fraction of mortar or concrete from the periphery of an air void; also known as Powers spacing factor. (See also **Philleo factor.**)

Spading—Consolidation of mortar or concrete by repeated insertion and withdrawal of a flat, spadelike tool.

Spall—A fragment, usually in the shape of a flake, detached from a larger mass by a blow, by the action of weather, by pressure, or by expansion within the larger mass; a small spall involves a roughly circular depression not greater than 20 mm in depth nor 150 mm in any dimension; a large spall may be roughly circular or oval or, in some cases elongated, more than 20 mm in depth and 150 mm in greatest dimension.

Spalling—The development of spalls.

Span—The distance between supports of a member.

Span length—See **Effective span.**

Spandrel—That part of a wall between the head of a window and the sill of the window above it.

Spandrel beam—A beam in the perimeter of a building, spanning between columns and usually supporting floors or roof.

Spatterdash—A rich mixture of portland cement and coarse sand; it is thrown onto a background by a trowel, scoop, or other appliance so as to form a thin, coarse-textured, continuous coating; as a preliminary treatment before rendering, it assists bond of the undercoat to the background, improves resistance to rain penetration, and evens out the suction of variable backgrounds. (See also **Parge** and **Dash-bond coat.**)

Specific gravity—The ratio of the mass of a unit volume of a material at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature.

1. **Apparent specific gravity**—The ratio of the mass in air of a unit volume of a material at a stated temperature to the mass in air of equal density of an equal volume of gas-free distilled water at a stated temperature. If the material is a solid, the volume is that of the impermeable portion.

2. **Bulk specific gravity**—The ratio of the mass in air of a unit volume of a permeable material (including both permeable and impermeable voids normal to the material) at a stated temperature to the mass in air of equal density of an equal volume of gas-free distilled water at a stated temperature.

3. **Bulk (saturated-surface-dry basis) specific gravity**—Same as bulk specific gravity except that the mass includes the water in the permeable voids.

See also **Absolute specific gravity.**

Specific gravity factor—The ratio of the weight of aggregates (including all moisture), as introduced into the mixer, to the effective volume displaced by the aggregates.

Specific heat—The amount of heat required per unit mass to cause a unit rise of temperature, over a small range of temperature.

Specific surface—The surface area of particles or of air voids contained in a unit weight or unit volume of a material; in the case of air voids in hardened concrete, the surface area of the air void volume expressed as square inches per cubic inch or square millimeters per cubic millimeter.

Spectrophotometer—Instrument for measuring intensity of radiant energy of desired frequencies absorbed by atoms of molecules; substances are analyzed by converting the absorbed energy to electrical signals, proportional to the intensity of radiation. (See also **Infrared spectroscopy** and **Flame photometer.**)

Spinning—The essential element of the process of producing spun concrete. (See also **Spun concrete.**)

Spiral reinforcement—Continuously wound reinforcement in the form of a cylindrical helix.

Spirally reinforced column—A column in which the vertical bars are enveloped by spiral reinforcement; i.e., closely spaced continuous hooping.

Splice—Connection of one reinforcing bar to another by lapping, welding, mechanical couplers, or other means; connection of welded wire fabric by lapping; connection of piles by mechanical couplers.

Split batch charging—Method of charging a mixer in which the solid ingredients do not all enter the mixer together; cement, and sometimes different sizes of aggregate, may be added separately.

Split block—See **Split-face block.**

Split-face block—Concrete masonry unit with one or more faces produced by purposeful fracturing of the unit, to provide architectural effects in masonry wall construction.

Splitting tensile strength—Tensile strength of concrete determined by a splitting tensile test.

Splitting tensile test (diametral compression test)—A test for tensile strength in which a cylindrical specimen is loaded to failure in diametral compression applied along the entire length.

Spray drying—A method of evaporating the liquid from a solution by spraying it into a heated gas.

Sprayed mortar—See **Shotcrete.**

Spread footing—A generally rectangular prism of concrete larger in lateral dimensions than the column or wall it supports, to distribute the load of a column or wall to the subgrade.

Spreader—A piece of lumber, usually about 1 x 2 in. (25 x 50 mm), cut to thickness of wall or other form and inserted to hold it temporarily at the correct dimension against tension of form ties; wires are usually attached to spreaders so they can be pulled up out of the forms as the pressure of concrete permits their removal; also a device consisting of reciprocating paddles, a revolving screw, or other mechanism for distributing concrete to required uniform thickness in a paving slab.

Spud vibrator—A vibrator used for consolidating concrete, having a vibrating casing or head, that is used by insertion into freshly placed concrete.

Spun concrete—Concrete compacted by centrifugal action, e.g., in the manufacture of pipes.

Stabilizer—A substance which makes a solution or suspension more stable, usually by keeping particles from precipitating.

Stage grouting—Sequential grouting of a hole in separate steps or stages in lieu of grouting the entire length at once.

Stalactite—A downward pointing deposit formed as an accretion of mineral matter produced by evaporation of dripping water from the surface of rock or of concrete, commonly shaped like an icicle. (See also **Stalagmite.**)

Stalagmite—An upward pointing deposit formed as an accretion of mineral matter produced by evaporation of dripping water, projecting from the surface of rock or concrete, commonly conical in shape. (See also **Stalactite.**)

Standard curing—Exposure of test specimens to specified conditions of moisture or humidity and of temperature. (See also **Fog curing.**)

Standard deviation—The root mean square deviation of individual values from their average.

Standard hook—A hook at the end of a reinforcing bar made in accordance with a standard.

Standard matched—Tongue-and-groove lumber with the tongue and groove offset rather than centered as in center matched lumber. (See also **Center matched.**)

Standard sand—Ottawa sand accurately graded to pass a U.S. Standard No. 20 (850-μm) sieve and be retained on a U.S. Standard No. 30 (600-μm) sieve, for use in the testing of cements. (See also **Ottawa sand** and **Graded standard sand.**)

Static load—The weight of a single stationary body or the combined weights of all stationary bodies in a structure (such as the load of a stationary vehicle on a roadway); or, during construction, the combined weight of forms, stringers, joists, reinforcing bars, and the actual concrete to be placed. (See also **Dead load.**)

Static modulus of elasticity—The value of Young's modulus of elasticity obtained by arbitrary criteria from measured stress-strain relationships derived from other than dynamic loading. (See also **Modulus of elasticity.**)

Stationary hopper—A container used to receive and temporarily store freshly mixed concrete.

Steam box—Enclosure for steam curing concrete products. (See also **Steam curing room.**)

Steam curing—Curing of concrete or mortar in water vapor at atmospheric or higher pressures and at temperatures between about 100 and 420 F (40 and 215 C). (See also **Atmospheric-pressure steam curing, Autoclave curing, Single-stage curing,** and **Two-stage curing.**)

Steam-curing cycle—The time interval between the start of the temperature-rise period and the end of the soaking period or the cooling-off period; also a schedule of the time and temperature of periods which make up the cycle.

Steam-curing room—A chamber for steam curing of concrete products at atmospheric pressure.

Steam kiln—See **Steam-curing room.**

Steel sheet—Cold-formed sheet or strip steel shaped as a structural member for the purpose of carrying the live and dead loads in lightweight concrete roof construction.

Steel trowel—See **Trowel.**

Stem bars—Bars used in the wall section of a cantilevered retaining wall or in the webs of a box; when a cantilevered retaining wall and its footing are considered as an integral unit, the wall is often referred to as the stem of the unit.

Stepped footing—A step-like support consisting of prisms of concrete of progressively diminishing lateral dimensions superimposed on each other to distribute the load of a column or wall to the subgrade.

Sticky cement—Finished cement which develops low or zero flowability during or after storage in silos, or after transportation in bulk containers, hopper-bottom cars, etc.; may be caused by: (a) interlocking of particles; (b) mechanical compaction; (c) electrostatic attraction between particles. (See also **Warehouse set.**)

Stiffback—See **Strongback.**

Stiffness—Resistance to deformation.

Stiffness factor—A measure of the stiffness of a structural member; for a prismatic member equal to the ratio of the product of the moment of inertia of the cross section and the modulus of elasticity for the material to the length of the member.

Stirrup—A reinforcement used to resist shear and diagonal tension stresses in a structual member; typically a steel bar bent into a U or box shape and installed perpendicular to or at an angle to the longitudinal reinforcement, and properly anchored; lateral reinforcement formed of individual units, open or closed, or of continuously wound reinforcement. (The term "stirrups" is usually applied to lateral reinforcement in flexural members and the term "ties" to lateral reinforcement in vertical compression members.) (See also **Tie.**)

Stockhouse set—See **Sticky cement** and **Warehouse set.**

Stone sand—Fine aggregate resulting from the mechanical crushing and processing of rock (See also **Sand** and **Fine aggregate.**)

Storage hopper—See **Stationary hopper.**

Straightedge—A rigid, straight piece of wood or metal used to strikeoff or screed a concrete surface to proper grade, or to check the planeness of a finished grade. (See also **Rod, Screed,** and **Strikeoff.**)

Straight-line theory—An assumption in reinforced-concrete analysis according to which the strains and stresses in a member under flexure are assumed to vary in proportion to the distance from the neutral axis.

Strain—Deformation of a material resulting from external loading, or the restrained portion of potential length and volume change resulting from internal causes.

Strain, unit—Deformation of a material expressed as the ratio of linear unit deformation to the distance within which that deformation occurs.

Strand—A prestressing tendon composed of a number of wires twisted about center wire or core.

Strand grip—A device used to anchor strands.

Stratification—The separation of overwet or overvibrated concrete into horizontal layers with increasingly lighter material toward the top; water, laitance, mortar, and coarse aggregate will tend to occupy successively lower positions in that order; a layered structure in concrete resulting from placing of successive batches that differ in appearance; occurrence in aggregate stockpiles of layers of differing grading or composition; a layered structure in a rock formation.

Stratling's compound—Dicalcium aluminate monosilicate-8-hydrate, a compound that has been found in reacted lime-pozzolan and cement-pozzolan mixtures.

Strength—See **Compressive strength, Fatigue strength, Flexural strength, Shear strength, Splitting tensile strength, Tensile strength, Ultimate strength,** and **Yield strength.**

Strength design method—A design method which requires service loads to be increased by specified load factors and computed theoretical strengths to be reduced by the specified phi (ϕ) factors.

Stress—Intensity of internal force (i.e., force per unit area) exerted by either of two adjacent parts of a body on the other across an imagined plane of separation; when the forces are parallel to the plane, the stress is called shear stress; when the forces are normal to the plane the stress is called normal stress; when the normal stress is directed toward the part on which it acts it is called compressive stress; when it is directed away from the part on which it acts it is called tensile stress.

Str

Stress corrosion—Corrosion of a metal initiated or accelerated by stress.

Stress relaxation—Stress loss developed when a constant length is maintained under stress at constant temperature.

Stressing end—In prestressed concrete, the end of the tendon from which the load is applied when tendons are stressed from one end only.

Stretcher—A masonry unit laid with its length horizontal and parallel with the face of a wall or other masonry member. (See also **Header.**)

Strike—See **Striking.**

Strikeoff—To remove concrete in excess of that which is required to fill the form evenly or bring the surface to grade; performed with a straightedged piece of wood or metal by means of a forward sawing movement or by a power operated tool appropriate for this purpose; also the name applied to the tool. (See also **Screed** and **Screeding.**)

Striking—The releasing or lowering of centering or other temporary support.

Stringer—A secondary flexural member which is parallel to the longitudinal axis of a bridge or other structure. (See also **Beam.**)

Strip—To remove formwork or a mold; also a long thin piece of wood, metal, or other material. (See also **Demolding.**)

Strip footing—See **Continuous footing.**

Strip foundation—A continuous foundation of which the length considerably exceeds the breadth.

Stripper—A liquid compound formulated to remove coatings by chemical or solvent action, or both.

Stripping—The removal of formwork or a mold. (See also **Demolding.**)

Strongback—A frame attached to the back of a form or precast structural member to stiffen or reinforce it during concrete placing operations or handling operations.

Structural concrete—See **Concrete, structural.**

Structural lightweight concrete—See **Concrete, structural lightweight.**

Strut—See **Shore.**

Stub wall—Low wall, usually 4 to 8 in. (100 to 200 mm) high, placed monolithically with concrete floor or other members to provide for control and attachment of wall forms; called **Kicker** in the United Kingdom.

Stucco—A cement plaster used for coating exterior walls and other exterior surfaces of buildings. (See also **Plaster.**)

Stud—Vertical member of appropriate size (2 x 4 to 4 x 10 in.) (50 x 100 to 100 x 250 mm) and spacing (16 to 30 in.) (400 to 750 mm) to support sheathing of concrete forms; also a headed steel device used to anchor steel plates or shapes to concrete members.

Subbase—A layer in a pavement system between the subgrade and base course or between the subgrade and a portland cement concrete pavement.

Subgrade—The soil prepared and compacted to support a structure or a pavement system.

Subgrade reaction—See **Contact pressure.**

Sub-purlin—A light structural section used as a secondary structural member; in lightweight concrete roof construction used to support the formboards over which the lightweight concrete is placed.

Sulfate attack—Chemical or physical reaction or both between sulfates usually in soil or ground water and concrete or mortar, primarily with calcium aluminate hydrates in the cement-paste matrix, often causing deterioration.

Sulfate resistance—Ability of concrete or mortar to withstand sulfate attack. (See also **Sulfate attack.**)

Sulfate-resistant cement—See **Cement, sulfate-resistant.**

Superimposed load—The load other than its own weight that is resisted by a structural member or system.

Super-sulfated cement—See **Cement, super-sulfated.**

Surface, specific—See **Specific surface.**

Surface active—Having the ability to modify surface energy and to facilitate wetting, penetrating, emulsifying, dispersing, solubilizing, foaming, frothing, etc., of other substances.

Surface area—See **Specific surface.**

Surface moisture—Free water retained on surfaces of aggregate particles and considered to be part of the mixing water in concrete, as distinguished from absorbed moisture.

Surface retarder—A retarder used by application to a form or to the surface of freshly mixed concrete to delay setting of the cement to facilitate construction joint cleanup or to facilitate production of exposed aggregate finish.

Surface tension—That property, due to molecular forces, that exists in the surface film of all liquids and tends to prevent the liquid from spreading.

Surfact texture—Degree of roughness or irregularity of the exterior surfaces of aggregate particles or hardened concrete.

Surface vibrator—A vibrator used for consolidating concrete by application to the top surface of a mass of freshly mixed concrete; four principal types exist: vibrating screeds, pan vibrators, plate or grid vibratory tampers, and vibratory roller screeds.

Surface voids—Cavities visible on the surface of a solid. (See also **Bugholes.**)

Surface water—See **Surface moisture.**

Surkhi—A pozzolan consisting of burned clay powder principally produced in India.

Sustained modulus of elasticity—Term including elastic and inelastic effects in one expression to aid in visualizing net effects of stress-strain up to any given time; computed by dividing the unit sustained stress by the sum of the elastic and inelastic deformations at that time. (See **Modulus of elasticity.**)

Swaybrace—A diagonal brace used to resist wind or other lateral forces. (See also **Bracing** and **X-brace.**)

Swelling—Volume increase caused by wetting or chemical changes, or both; a function of time but not of temperature or of stress due to external load.

Swift—A reel or turntable on which prestressing tendons are placed to facilitate handling and placing.

Swirl finish—A nonskid texture imparted to a concrete surface during final troweling by keeping the trowel flat and using a rotary motion.

Swiss hammer—See **Rebound hammer.**

Syneresis—The contraction of a gel, usually evidenced by the separation from the gel of small amounts of liquid; a process possibly significant in bleeding and cracking of fresh portland cement mixtures.

Syngenite—Potassium calcium sulfate hydrate, a compound sometimes produced during hydration of portland cement, found in deteriorating portland-cement concrete and said to form in portland cement during storage by reaction of potassium sulfate and gypsum.

Systems building—See **Industrialized building.**

T

Talc—A mineral with a greasy or soapy feel, very soft, having the composition $Mg_3Si_4O_{10}(OH)_2$. (See also **Cement, masonry.**)

Tamper—

1. An implement used to consolidate concrete or mortar in molds or forms.

2. A hand-operated device for compacting floor topping or other unformed concrete by impact from the dropped device in preparation for strikeoff and finishing; contact surface often consists of a screen or a grid of bars to force coarse aggregates below the surface to prevent interference with floating or trowelling.

See also **Jitterbug.**

Tamping—The operation of compacting freshly placed concrete by repeated blows or penetrations with a tamping device.

Tamping rod—See **Rod, tamping.**

Tangent modulus—See **Modulus of elasticity.**

T-beam—A beam composed of a stem and a flange in the form of a "T."

Telltale—Any device designed to indicate movement of formwork or a point along the length of a pile under load.

Temperature cracking—Cracking due to tensile failure, caused by temperature drop in members subjected to external restraints or temperature differential in members subjected to internal restraints.

Temperature reinforcement—Reinforcement designed to carry stresses resulting from temperature changes; also the minimum reinforcement for areas of members which are not subjected to primary stresses or necessarily to temperature stresses.

Temperature rise—The increase of temperature caused by absorption of heat or internal generation of heat, as by hydration of cement in concrete.

Temperature rise period—The time interval during which the temperature of a concrete product rises at a controlled rate to the desired maximum in autoclave or atmospheric-pressure steam curing.

Temperature stress—Stress in a structure or a member due to changes or differentials in temperature in the structure or member.

Tempering—The addition of water and mixing of concrete or mortar as necessary to bring it to the desired consistency during the prescribed mixing period; for truck-mixed concrete this will include any addition of water as may be necessary to bring the load to the correct slump on arrival at the work site but not after a period of waiting to discharge the concrete.

Template—A thin plate or board frame used as a guide in positioning or spacing form parts, reinforcement, or anchors; also a full-size mold, pattern or frame, shaped to serve as a guide in forming or testing contour or shape.

Temporary stress—A stress which may be produced in a precast concrete member or component of a precast concrete member during fabrication or erection, or in cast-in-place concrete structures due to construction or test loadings.

Tendon—A steel element such as a wire, cable, bar, rod, or strand used to impart prestress to concrete when the element is tensioned.

Tendon profile—The path or trajectory of the prestressing tendon.

Tensile strength—Maximum unit stress which a material is capable of resisting under axial tensile loading, based on the cross-sectional area of the specimen before loading.

Tensile stress—Stress resulting from tension.

Tension reinforcement—Reinforcement designed to carry tensile stresses such as those in the bottom of a simple beam.

Terrazzo concrete—See **Concrete, terrazzo.**

Tesserae—Small pieces of marble tile or glass used in mosaics.

Test—A trial, examination, observation, or evaluation used as a means of measuring a physical or chemical characteristic of a material, or a physical characteristic of a structural element or a structure.

Testing machine—A device for applying test conditions and accurately measuring results.

Tetracalcium aluminoferrite—A compound in the calcium aluminoferrite series, having the composition $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$, abbreviated C_4AF, which is usually assumed to be the aluminoferrite present when compound calculations are made from the results of chemical analysis of portland cement. (See also **Brown-millerite.**)

Texture—The pattern or configuration apparent in an exposed surface, as of concrete or mortar, including roughness, streaking, striation, or departure from flatness.

Texturing—The process of producing a special texture on unhardened or hardened concrete.

T-head—In precast framing, a segment of girder crossing the top of an interior column; also the top of a shore formed with a braced horizontal member projecting on two sides forming a T-shaped assembly.

Thermal conductivity—A property of a homogeneous body measured by the ratio of the steady state heat flux (time rate of heat flow per unit area) to the temperature gradient (temperature difference per unit length of heat flow path) in the direction perpendicular to the area.

Thermal diffusivity—Thermal conductivity divided by the product of specific heat and unit weight; an index of the facility with which a material undergoes temperature change.

Thermal shock—The subjection of a material or body, such as partially hardened concrete, to a rapid change in temperature which may be expected to have a potentially deleterious effect.

Thermoplastic—Becoming soft when heated and hard when cooled.

Thermosetting—Recoming rigid by chemical reaction and not remeltable.

Thin-shell precast—Precast concrete characterized by thin slabs and web sections. (See also **Shell construction.**)

Thixotropy—The property of a material that enables it to stiffen in a short period on standing, but to acquire a lower viscosity on mechanical agitation, the process being reversible; a material having this property is termed thixotropic or shear thinning. (See **Rheology.**)

Threaded anchorage—An anchorage device which is provided with threads to facilitate attaching the jacking device and to effect the anchorage.

Tie—

1. Loop of reinforcing bars encircling the longitudinal steel in columns.

2. A tensile unit adapted to holding concrete forms secure against the lateral pressure of unhardened concrete, with or without provision for spacing the forms a definite distance apart, and with or without provision for removal of metal to a specified distance from the finished concrete surface.

Tie bar—Bar at right angles to and tied to minimum reinforcement to keep it in place; bar extending across a construction joint.

Tie rod—See **Form tie** and **Tieback.**

Tieback—A rod fastened to a deadman, a rigid foundation, or a rock or soil anchor to prevent lateral movement of formwork, sheet pile walls, retaining walls, bulkheads, etc.

Tied column—A column laterally reinforced with ties.

Tiers—See **Lifts.**

Tilting concrete mixer—See **Mixer, tilting.**

Tilt-up—A method of concrete construction in which members are cast horizontally at a location adjacent to their eventual position, and tilted into place after removal of molds.

Time-dependent deformation—Combined effects of autogenous volume change, contraction, creep, expansion, shrinkage, and swelling occurring during an appreciable period of time; not synonymous with **Inelastic behavior** or **Volume change.**

Time of haul—In production of ready-mixed concrete, the period from first contact between mixing water and cement until completion of discharge of the freshly mixed concrete.

Time of setting—See **Initial setting time** and **Final setting time.**

Time of set—See **Initial setting time** and **Final setting time.**

Tobermorite—A mineral found in Northern Ireland and elsewhere, having the approximate formula $Ca_4(Si_6O_{18}H_2)$. $Ca \cdot 4H_2O$ identified approximately with the artificial product tobermorite (G) of Brunauer, a hydrated calcium silicate having CaO SiO_2 ratio in the range 1.39 to 1.75 and forming minute layered crystals that constitute the principal cementing medium in portland cement concrete; a mineral with 5 mols of lime to 6 mols of silica, usually occurring in plate like crystals, which is easily synthesized at steam pressures of about 100 psig and higher; the binder in several properly autoclaved products.

Tobermorite gel—The binder of concrete cured moist or in atmospheric-pressure steam, a lime-rich gel-like solid containing 1.5 to 2.0 mols of lime per mol of silica.

Toenail—To drive a nail at an angle.

Tolerance—

1. The permitted variation from a given dimension or quantity.

2. The range of variation permitted in maintaining a specified dimension.

3. A permitted variation from location or alignment.

Tom—See **Shore.**

Tongue and groove—A type of lumber or precast concrete pile having mated projecting and grooved edges to provide a tight fit, abbreviated "T & G."

Top form—Form required on the upper or outer surface of a sloping slab or thin shell.

Topping—

1. A layer of concrete or mortar placed to form a floor surface on a concrete base.

2. A structural, cast-in-place surface for precast floor and roof systems.

3. The mixture of marble chips and matrix which, when properly processed, produces a terrazzo surface.

Torque viscometer—An apparatus used for measuring the consistency of slurries in which the energy required to rotate a device suspended in a rotating cup is proportional to viscosity.

Toughness—The property of matter which resists fracture by impact or shock.

Tower—A composite structure of frames, braces, and accessories.

Trajectory of prestressing force—The path along which the prestress is effective in a structure or member; it is coincident with the center of gravity of the tendons for simple flexural members and statically indeterminate members which are prestressed with concordant tendons, but is not coincident with the center of gravity of the tendons of a statically indeterminate structure which is prestressed with nonconcordant tendons.

Transfer—The act of transferring the stress in prestressing tendons from the jacks or pretensioning bed to the concrete member.

Transfer bond—In pretensioning, the bond stress resulting from the transfer of stress resulting from the tendon to the concrete.

Transfer length—See **Transmission length.**

Transfer strength—The concrete strength required before stress is transferred from the stressing mechanism to the concrete.

Transformed section—A hypothetical section of one material arranged so as to have the same elastic properties as a section of two materials.

Transit-mixed concrete—Concrete, the mixing of which is wholly or principally accomplished in a truck mixer.

Translucent concrete—See **Concrete, translucent.**

Transmission length—The distance at the end of a pretensioned tendon necessary for the bond stress to develop the maximum tendon stress; sometimes called **Transfer length.**

Transverse cracks—Cracks that develop at right angles to the long direction of the member.

Transverse joint—A joint parallel to the intermediate dimension of a structure.

Transverse prestress—Prestress that is applied at right angles to the principal axis of a member.

Transverse reinforcement—Reinforcement at right angles to the principal axis of a member.

Transverse strength—See **Flexural strength** and **Modulus of rupture.**

Traprock—Any of various fine-grained, dense, dark colored igneous rocks, typically basalt or diabase; also called "trap."

Trass—A natural pozzolan of volcanic origin found in Germany.

Traveler—An inverted-U-shaped structure usually mounted on tracks which permit it to move from one location to another to facilitate the construction of an arch, bridge, or building.

Tremie—A pipe or tube through which concrete is deposited under water, having at its upper end a hopper for filling and a bail for moving the assemblage.

Tremie concrete—Subaqueous concrete placed by means of a tremie.

Tremie seal—The depth to which the discharge end of the tremie pipe is kept embedded in the fresh concrete that is being placed; a layer of tremie concrete placed in a cofferdam for the purpose of preventing the intrusion of water when the cofferdam is dewatered.

Trial batch—A batch of concrete prepared to establish or check proportions of the constituents.

Triaxial compression test—A test in which a specimen is subjected to a confining hydrostatic pressure and then loaded axially to failure.

Triaxial test—A test in which a specimen is subjected simultaneously to lateral and axial loads.

Tricalcium aluminate—A compound having the composition $3CaO \cdot Al_2O_3$, abbreviated C_3A.

Tricalcium silicate—A compound having the composition $3CaO \cdot SiO_2$, abbreviated C_3S, an impure form of which (alite) is a main constituent of portland cement. (See also **Alite.**)

Trough mixer—See **Open-top mixer.**

Trowel—A flat, broad-blade steel hand tool used in the final stages of finishing operations to impart a relatively smooth surface to concrete floors and other unformed concrete surfaces; also a flat triangular-blade tool used for applying mortar to masonry.

Trowel finish—The smooth finish surface produced by troweling.

Troweling—Smoothing and compacting the unformed surface of fresh concrete by strokes of a trowel.

Troweling machine—A motor driven device which operates orbiting steel trowels on radial arms from a vertical shaft.

Truck-mixed concrete—See **Transit-mixed concrete.**

Truck mixer—A concrete mixer suitable for mounting on a truck chassis and capable of mixing concrete in transit. (See also **Horizontal-axis mixer, Inclined-axis mixer, Open-top mixer,** and **Agitator.**)

T-shore—A shore with a T-head.

Tub mixer—See **Open-top mixer.**

Tube-and-coupler shoring—A load-carrying assembly of tubing or pipe which serves as posts, braces, and ties, a base supporting the posts, and special couplers which connect the uprights and join the various members.

Turbidimeter—A device for measuring the particle-size distribution of a finely divided material by taking successive measurements of the turbidity of a suspension in a fluid.

Turbidimeter fineness—The fineness of a material such as portland cement, usually expressed as total surface area in square centimeters per gram, as determined with a turbidimeter. (See also **Wagner fineness.**)

Turbine mixer—See **Open-top mixer.**

Twin-twisted reinforcement—See **Reinforcement, twin-twisted bar.**

Two-stage curing—A process in which concrete products are cured in low-pressure steam, stacked, and then autoclaved.

Two-way reinforced footing—A footing having reinforcement in two directions generally perpendicular to each other.

Two-way system—A system of reinforcement: bars, rods, or wires placed at right angles to each other in a slab and intended to resist stresses due to bending of the slab in two directions.

U

Ultimate design resisting moment—The moment at which a section reaches its ultimate usable strength, most commonly the moment at which the tensile reinforcement reaches its specified yield strength.

Ultimate load—The maximum load which may be placed on a structure before its failure due to buckling of column members or failure of some component; also the load at which a unit or structure fails.

Ultimate moment—The bending moment at which a section reaches it ultimate usable strength, most commonly the moment at which the tensile reinforcement reaches its specified yield strength.

Ultimate shear stress—The stress at a section which is loaded to its maximum in shear. (See also **Shear strength.**)

Ultimate strength—The maximum resistance to load that a member or structure is capable of developing before failure occurs; or, with reference to cross sections of members, the largest moment, axial force, torsion or shear a structural concrete cross section will support.

Ultimate strength design—See **Strength design method.**

Unbonded member—Post-tensioned, prestressed concrete element in which tensioning force is applied against end anchorages only, tendons being free to move within the elements.

Unbonded post-tensioning—Post-tensioning in which the tendons are not grouted after stressing.

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

Unbraced length of column—Distance between adequate lateral supports.

Underbed—The base mortar, usually horizontal, into which strips are embedded and on which terrazzo topping is applied.

Undersanded—With respect to concrete, containing an insufficient proportion of fine aggregate to produce optimum properties in the fresh mixture, especially workability and finishing characteristics.

Unit water content—The quantity of water per unit volume of freshly mixed concrete, often expressed as pounds or gallons per cubic yard; the quantity of water on which the water-cement ratio is based, not including water absorbed by the aggregate.

Unit weight—See **Bulk density** and **Specific gravity.**

Unprotected corner—Corner of a slab with no adequate provision for load transfer, so that the corner must carry over 80 percent of the load. (See also **Protected corner.**)

Unreinforced concrete—See **Plain concrete.**

Unsound—Not firmly made, placed, or fixed; subject to deterioration or disintegration during service exposure.

V

Vacuum concrete—Concrete from which water and entrapped air are extracted by a vacuum process before hardening occurs.

Vacuum saturation—A process for increasing the amount of filling of the pores in a porous material, such as lightweight aggregate, with a fluid, such as water, by subjecting the porous material to reduced pressure in the presence of the fluid.

Valve bag—Paper bag for cement or other material, either glued or sewn, made of four or five plies of kraft paper and completely closed except for a self-sealing paper valve through which the contents are introduced.

Vapor barrier—Waterproof membrane placed under concrete floor slabs that are placed on grade.

Vapor pressure—A component of atmospheric pressure which is caused by the presence of vapor; expressed in inches, centimeters, or millimeters of height of a column of mercury.

Vehicle—Liquid carrier or binder of solids.

Veneer—A masonry facing which is attached to the backup but not so bonded as to act with it under load.

Venetian—A type of terrazzo topping in which large chips are incorporated.

Vent pipe—A small-diameter pipe used in concrete construction to permit escape of air in a structure being concreted or grouted.

Vented form—A form so constructed as to retain the solid constituents of concrete and permit the escape of water and air.

Vermiculite—A group name for certain platy minerals, hydrous silicates of aluminum, magnesium, and iron; characterized by marked exfoliation on heating; also a constituent of clays.

Vermiculite concrete—Concrete in which the aggregate consists of exfoliated vermiculite.

Vibrated concrete—Concrete compacted by vibration during and after placing.

Vibration—Energetic agitation of freshly mixed concrete during placement by mechanical devices either pneumatic or electric, that create vibratory impulses of moderately high frequency that assist in consolidating the concrete in the form or mold.

1. **External vibration** employs vibrating devices attached at strategic positions on the forms and is particularly applicable to manufacture of precast items and for vibration of tunnel-lining forms; in manufacture of concrete products, external vibration or impact may be applied to a casting table.

2. **Internal vibration** employs one or more vibrating elements that can be inserted into the concrete at selected locations, and is more generally applicable to in-place construction.

3. **Surface vibration** employs a portable horizontal platform on which a vibrating element is mounted.

Vibration limit—That time at which fresh concrete has hardened sufficiently to prevent its becoming mobile when subjected to vibration.

Vibrator—An oscillating machine used to agitate fresh concrete so as to eliminate gross voids, including entrapped air but not entrained air, and produce intimate contact with form surfaces and embedded materials.

Vicat apparatus—A penetration device used in the testing of hydraulic cements and similar materials.

Vicat needle—A weighted needle for determining setting time of hydraulic cements.

Viscometer—Instrument for determining viscosity of slurries, mortars, or concretes.

Viscosity—A property of a material which resists change in the shape or arrangement of its elements during flow, and the measure there of.

Visual concrete—See **Exposed concrete.**

Void-cement ratio—Volumetric ratio of air plus net mixing water to cement in a concrete or mortar mixture.

Volatile material—Material that is subject to release as a gas or vapor; liquids that evaporate readily.

Volume batching—The measuring of the constituent materials for mortar or concrete by volume.

Volume change—An increase or decrease in volume. (See also **Deformation.**)

W

Waffle—See **Dome.**

Wagner fineness—The fineness of portland cement, expressed as total surface area in square centimeters per gram, determined by the Wagner turbidimeter apparatus and procedure.

Wale—A long horizontal formwork member (usually double) used to hold studs in place; also called **Waler** or **Ranger.**

Wall—A vertical element used primarily to enclose or separate spaces.

Wall form—A retainer or mold so erected as to give the necessary shape, support and finish to a concrete wall.

Warehouse set—The partial hydration of cement stored for a time and exposed to atmospheric moisture, or mechanical compaction occurring during storage.

Warping—A deviation of a slab or wall surface from its original shape, usually caused by temperature or moisture differentials or both within the slab or wall. (See also **Curling.**)

Warping joint—A joint with the sole function of permitting warping of pavement slabs when moisture and temperature differentials occur in the pavement, i.e., longitudinal or transverse joints with bonded steel or tie bars passing through them.

Wash (or **flush**) **water**—Water carried on a truck mixer in a special tank for flushing the interior of the mixer after discharge of the concrete.

Water gain—See **Bleeding.**

Water-cement ratio—The ratio of the amount of water, exclusive only of that absorbed by the aggregates, to the amount of cement in a concrete or mortar mixture; preferably stated as a decimal by weight.

"Waterproofed" cement—Cement interground with a water repellent material such as calcium stearate.

"Waterproofing" compound—See **Compound, "waterproofing."**

Water-reducing agent—A material which either increases slump of freshly mixed mortar or concrete without increasing water content or maintains workability with a reduced amount of water, the effect being due to factors other than air entrainment.

"Water-repellent" cement—A hydraulic cement having a water-repellent agent added during the process of manufacture, with the intention of resisting the absorption of water by the concrete or mortar.

Water ring—Perforated manifold in nozzle of dry-mix shotcrete equipment through which water is added to the materials.

Waterstop—A thin sheet of metal, rubber, plastic, or other material inserted across a joint to obstruct the seeping of water through the joint.

Weakened-plane joint—See **Groove joint.**

Wearing course—A topping or surface treatment to increase the resistance of a concrete pavement or slab to abrasion.

Weathering—Changes in color, texture, strength, chemical composition or other properties of a natural or artificial material due to the action of the weather.

Web bar—See **Web reinforcement.**

Web reinforcement—Reinforcement placed in a concrete member to resist shear and diagonal tension.

Wedge—A piece of wood or metal tapering to a thin edge, used to adjust elevation or tighten formwork.

Wedge anchorage—A device for providing the means of anchoring a tendon by wedging.

Weight batching—Measuring the constituent materials for mortar or concrete by weight.

Welded butt splice—A reinforcing bar splice made by welding the butted ends.

Welded-wire fabric—A series of longitudinal and transverse wires arranged substantially at right angles to each other and welded together at all points of intersection.

Welded-wire fabric reinforcement—Welded-wire fabric in either sheets or rolls, used to reinforce concrete.

Well-graded aggregate—Aggregate having a particle size distribution which will produce maximum density, i.e., minimum void space.

Wet process—In the manufacture of cement, the process in which the raw materials are ground, blended, mixed, and pumped while mixed with water; the wet process is chosen where raw materials are extremely wet and sticky, which would make drying before crushing and grinding difficult. (See also **Dry process.**)

Wet screening—Screening to remove from fresh concrete all aggregate particles larger than a certain size.

Wet-mix shotcrete—Shotcrete wherein all ingredients, including mixing water, are mixed before introduction into the delivery hose; it may be pneumatically conveyed or moved by displacement. (See also **Pneumatic feed** and **Positive displacement.**)

Wet sieving—Use of water during sieving of a material on a No. 200 ($75\,\mu$m) or No. 325 ($45\,\mu$m) sieve.

Wettest stable consistency—The condition of maximum water content at which cement grout or mortar will adhere to a vertical surface without sloughing.

Wetting agent—A substance capable of lowering the surface tension of liquids, facilitating the wetting of solid surfaces and permitting the penetration of liquids into the capillaries.

Wheel load—The portion of the gross weight of a loaded vehicle transferred to a supporting structure under a given wheel of the vehicle.

White cement—See **Cement, white.**

Wing pile—A bearing pile, usually of concrete, widened in the upper portion to form part of a sheet pile wall.

Wire, cold-drawn—Wire made from the rods hot rolled from billets and then cold-drawn through dies. (See also **Reinforcement, cold-drawn wire.**)

Wire mesh—See **Welded-wire fabric.**

Wire winding—Application of high tensile wire, wound under tension by machines, around circular concrete or shotcrete walls, domes, or other tension resisting structural components.

Wobble coefficient—A coefficient used in determining the friction loss occurring in post-tensioning, which is assumed to account for the secondary curvature of the tendons.

Wobble friction—In prestressed concrete, the friction caused by the unintended deviation of the prestressing sheath or duct from its specified profile.

Workability—That property of freshly mixed concrete or mortar which determines the ease and homogencity with which it can be mixed, placed, compacted, and finished.

Working load—Forces normally imposed on a member in service.

Working stress—Maximum permissible design stress using working stress design methods.

Working stress design—A method of proportioning structures or members for prescribed working loads at stresses well below the ultimate, and assuming linear distribution of flexural stresses.

Woven-wire fabric—A prefabricated steel reinforcement composed of cold-drawn steel wires mechanically twisted together to form hexagonally shaped openings.

Woven-wire reinforcement—See **Welded-wire fabric.**

Wrecking strip—Small piece or panel fitted into a formwork assembly in such a way that it can be easily removed ahead of main panels or forms, making it easier to strip those major form components.

Wythe (leaf)—Each continuous vertical section of a wall one masonry unit in thickness.

X

X-brace—Paired set of sway braces.

Xonotlite—5-calcium-5-silicate monohydrate (C_5S_5H), a natural mineral that is readily synthesized at 150 to 350 C under saturated steam pressure; a constituent of sand-lime masonry units.

X-ray diffraction—The diffraction of x-rays by substances having a regular arrangement of atoms; a phenomenon used to identify substances having such structure.

X-ray fluorescence—Characteristic secondary radiation emitted by an element as a result of excitation by x-rays, used to yield chemical analysis of a sample.

Y

Yellowing—Development of yellow color or cast in white or clear coatings, on aging.

Yield—The volume of freshly mixed concrete produced from a known quantity of ingredients; the total weight of ingredients divided by the unit weight of the freshly mixed concrete; also, the number of product units, such as block, produced per bag of cement or per batch of concrete.

Yield point—That point during increasing stress when the proportion of stress to strain becomes substantially less than it has been at smaller values of stress.

Yield strength—The stress, less than the maximum attainable stress, at which the ratio of stress to strain has dropped well below its value at low stresses, or at which a material exhibits a specified limiting deviation from the usual proportionality of stress to strain.

Yok

Yoke—A tie or clamping device around column forms or over the top of wall or footing forms to keep them from spreading because of the lateral pressure of fresh concrete; also part of a structural assembly for slipforming which keeps the forms from spreading and transfers form loads to the jacks.

Young's modulus—See **Modulus of elasticity.**

Z

Zero-slump concrete—Concrete of stiff or extremely dry consistency showing no measurable slump after removal of the slump cone. (See also **Slump** and **No-slump concrete.**)

ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction, and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be incorporated directly into the Project Documents.

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Guide to Durable Concrete

Reported by ACI Committee 201

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This guide is essentially an update of the committee report "Durability of Concrete in Service" which appeared in the December 1962 ACI JOURNAL. There are a number of major revisions reflecting increased knowledge of the subject.

A separate chapter is devoted to each of the main types of concrete deterioration. Their mechanism is described and the requirements for materials, design, and construction procedures necessary to prevent damage to the concrete are given. A selected bibliography is included with each chapter.

Keywords: abrasion; **abrasion resistance**; acid resistance; adhesives; admixtures; **aggregates**; air entrainment; **alkali-aggregate reactions**; **alkali-carbonate reactions**; bridge decks; calcium chlorides; **cement-aggregate reactions**; cement pastes; chemical analysis; **chemical attack**; chlorides; **coatings**; **concrete durability**; concrete pavements; **corrosion**; **corrosion resistance**; cracking (fracturing); damage; **deicers**; deterioration; **durability**; epoxy resins; floors; **freeze-thaw durability**; freezing; petrography; plastics, polymers, and resins; **protective coatings**; reinforced concrete; reinforcing steels; **repairs**; skid resistance; spalling; **sulfate attack**; waterproof coatings.

FOREWORD

ACI Committee 201 was organized in 1957, and published a report "Durability of Concrete in Service" in the December 1962 ACI Journal.

The committee has also published a "Guide for Making a Condition Survey of Concrete in Service" in the November 1968 ACI Journal, and a symposium volume, Durability of Concrete, SP-47, in 1975.

Charles F. Scholer was chairman of Committee 201 during the early development of this guide.

*Deceased

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INTRODUCTION

Durability of portland cement concrete is defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration. Durable concrete will retain its original form, quality, and serviceability when exposed to its environment. Some excellent general references on the subject are available.[1-3]*

This report discusses in some depth the more important causes of concrete deterioration, and gives recommendations on how to prevent such damage. Chapters are included on freezing and thawing, aggressive chemical exposure, abrasion, reactive aggregates, corrosion of embedded materials, repair methods, and the use of coatings to enhance durability. The fire resistance of concrete is not covered, since it is included in the mission of ACI Committee 216.

Freezing and thawing damage is a serious problem in northern climates, and the mechanisms involved are now fairly well understood. In pavements the damage is greatly accelerated by the use of deicing salts, often resulting in severe

*References are listed at the end of each chapter.

scaling at the surface. Fortunately, concrete made with good aggregates, low water-cement ratio, and air entrainment will have good resistance to cyclic freezing.

By using a suitable cement and a properly proportioned mix, concrete will resist sulfates in soil, ground water, or seawater. High quality concrete will resist mild acid attack, but no concrete offers good resistance to attack by strong acids; special protection is necessary in this case.

Sometimes concrete surfaces will wear away as the result of abrasive action. Wear can be a particular problem in industrial floors. In hydraulic structures, particles of sand or gravel in flowing water can erode surfaces. The use of high quality concrete and, in extreme cases, a very hard aggregate will usually result in adequate durability under these exposures. The recent use of studded tires on automobiles has caused serious wear in concrete pavements; conventional concrete will not withstand this action.

The spalling of concrete in bridge decks has become a serious problem in recent years. The principal cause is corrosion of the reinforcing steel, which is largely due to the use of deicing salts. The corrosion products produce an expansive force which causes the concrete to spall out above the steel. Ample cover over the steel and use of a low-permeability, air-entrained concrete will assure good durability in the great majority of cases, but more positive protection is needed for very severe exposures.

Although aggregate is commonly considered to be an inert filler in concrete, such is not always the case. Certain aggregates can react with portland cement, causing expansion and deterioration. Fortunately, care in the selection of aggregate sources, and use of low-alkali cement and pozzolans where appropriate, will prevent this problem.

The final chapters of this report discuss the repair of concrete which has not withstood the forces of deterioration, and the use of protective coatings to enhance durability.

The committee wishes to stress that good design and materials alone will not assure durable concrete. Good quality control and workmanship are absolutely essential to the production of durable concrete. Experience has shown that two areas should receive special attention: (1) control of entrained air and (2) finishing of slabs. The *ACI Manual of Concrete Inspection* describes good concrete practices and inspection procedures.[4]

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2. ACI Committee 201, "Guide for Making a Condition Survey of Concrete in Service," ACI Journal, *Proceedings* V. 65, No. 11, Nov. 1968, pp. 905-918.

3. Woods, Hubert, *Durability of Concrete Construction*, Monograph No. 4, American Concrete Institute/Iowa State University Press, Detroit, 1968, 190 pp.

4. ACI Committee 311, *ACI Manual of Concrete Inspection*, SP-2, 6th Edition, American Concrete Institute, Detroit, 1975, 268 pp.

CHAPTER 1—FREEZING AND THAWING

1.1—General

Exposing damp concrete to freezing and thawing cycles is a severe test of the material, and poor concrete will certainly fail. On the other hand, air-entrained concrete which is properly proportioned, manufactured, placed, finished, and cured will almost always resist cyclic freezing for many years.

It should be recognized, however, that even good concrete may suffer damage from cyclic freezing in unusual conditions, particularly concrete which is kept in a state of nearly complete saturation. Also, in cases where the concrete is saturated on the back side and exposed to air on the front side, it may exhibit extremely variable behavior, ranging from complete freedom from damage to total failure.

A good general discussion on the subject of frost action in concrete is provided by Cordon.[1]

1.2—Mechanism of frost action

Powers and his associates conducted extensive research on frost action in concrete from 1933 to 1961. They were able to develop reasonable hypotheses to explain the rather complex mechanisms involved.

Hardened cement paste and aggregate behave quite differently when subjected to cyclic freezing, and it is generally agreed that each must be considered separately.

1.2.1 *Freezing in cement paste*—In his early papers, Powers[2-5] attributed frost damage in cement paste to stresses caused by hydraulic pressure in the pores, the pressure being due to resistance to movement of water away from the regions of freezing. It was believed that the magnitude of the pressure depended on the rate of freezing, degree of saturation and coefficient of permeability of the paste, and the length of the flow-path to the nearest place for the water to escape. The benefits of entrained air were explained in terms of the shortening of flow-paths to places of escape. Some authorities still accept this hypothesis.

Later studies by Powers and Helmuth produced strong evidence that the hydraulic pressure hypotheses are not consistent with the experimental results.[6] They found that during freezing of cement paste most of the water movement is toward, not away from, sites of freezing as had been previously believed. Also, the dilations (expansions) during freezing generally decreased with increased rate of cooling.[7-9] Both of these findings were contrary to the hydraulic pressure hypothesis, and indicated that a modified form of a theory previously advanced by Collins[10] (originally developed to explain frost action in soil) is applicable.

Powers and Helmuth point out that the water in cement paste is in the form of a weak alkali solution. When the temperature of the concrete drops below the freezing point, there will be an initial period of supercooling, after which ice crystals will form in the larger capillaries. This results in an increase in alkali content in the unfrozen portion of the solution in these capillaries, creating an osmotic potential which impels water molecules in the nearby unfrozen pores to begin diffusing into the solution in the frozen cavities. The resulting dilution of the solution in contact with the ice allows further growth of the body of ice (ice-accretion). When the cavity becomes full of ice and solution, any further ice-accretion produces dilative pressure which can cause the paste to fail. When water is being drawn out of unfrozen capillaries, the paste tends to shrink. (Experiments have verified that shrinkage of paste, or concrete, occurs during part of the freezing cycle.)

According to Powers, when the paste contains entrained air, and the average distance between air bubbles is not too great, the bubbles compete with the capillaries for the unfrozen water and normally win this competition. For a better understanding of the mechanism involved, the reader is directed to the references cited above. Many researchers now believe that stresses resulting from osmotic pressure cause most of the frost damage to cement paste.

In recent years, Litvan[11] has further studied frost action in cement paste. Litvan believes the water adsorbed on the surface or contained in the pores and which cannot freeze plays a major role. Because of the difference in vapor pressure of this supercooled liquid and the bulk ice in the surroundings of the paste system, there will be migration of water to locations where it is able to freeze, such as the larger pores or the outer surface. The process leads to partial desiccation of the paste and accumulation of ice in crevices and cracks. Failure occurs when the required redistribution of water cannot take place in an orderly fashion either because the amount of water is too large (high water-cement ratio for the same degree of saturation), the available time is too short (rapid cooling), or the path of migration is too long (lack of entrained air bubbles). Litvan believes that in such cases, the freezing forms a semi-amorphous solid (not ice crystals) resulting in great internal stresses.

There is general agreement that cement paste can be made completely immune to damage from freezing temperatures by means of entrained air, unless special exposure conditions result in filling of the air voids. However, air entrainment alone does not preclude the possibility of damage of concrete due to freezing; freezing phenomena in aggregate particles must also be taken into consideration.

1.2.2 *Freezing in aggregate particles*—Most rocks have pore sizes much larger than those in cement paste, and Powers[2] found that they expel water during freezing. The committee believes the hydraulic pressure theory, previously described for cement paste, is applicable in most cases.

Dunn and Hudec[12] advanced the "ordered water" theory, which states that the principal cause of deterioration of rock is not freezing but the expansion of adsorbed water (which is not freezable); specific cases of failure without freezing of claybearing limestone aggregates seemed to support this conclusion. This, however, is not consistent with the results of research by Helmuth[13] who found that adsorbed water does not expand, but actually contracts during cooling. Nevertheless, Helmuth agrees that the adsorption of large amounts of water in aggregates having a very fine pore structure can disrupt concrete (but through ice formation).

The size of the coarse aggregate has been shown to be an important factor in its frost resistance. Verbeck and Landgren[14] have demonstrated that for any given natural rock, frozen unconfined by cement paste, there is a critical size below which it can be frozen without damage. They showed that the critical size of rocks of good quality range upwards from perhaps a quarter of an inch. However, some aggregates (e.g., granite, basalt, diabase, quartzite, marble) have capacities for freezable water so low that they do not produce stress when freezing occurs—regardless of the particle size.

The role of entrained air in alleviating the effect of freezing in rock particles is minimal.

1.2.3 *Overall effects in concrete*—Without entranied air, the paste matrix surrounding the aggregate particles may fail when it becomes critically saturated and is frozen. However, if the matrix contains an appropriate distribution of entrained air voids characterized by a spacing factor less than about 0.008 in. (0.20 mm), freezing does not produce destructive stress.[15]

There are some rocks which contain practically no freezable water. Air-entrained concrete made with an aggregate composed entirely of such rocks will withstand freezing even under continuously wet exposures for a long time. This time may be shortened if the air voids fill with water and solid matter.

If absorptive aggregates (such as structural lightweight) are used and the concrete is in a continuously wet environment, it will probably fail if the coarse aggregate becomes saturated. The pressure developed when the particles expel water during freezing ruptures the particles and the matrix. If the particle is near the concrete surface, a popout can result.

Normally, aggregate in concrete is not in a critical state of saturation at the end of the construction period because of desiccation produced by the chemical reaction during hardening (self-desiccation of the cement paste) and loss by evaporation. Therefore, if any of the aggregate ever becomes critically saturated, it will be by water obtained from an outside source later on. Yet structures so situated that all exposed surfaces are kept continuously wet, and yet are periodically subject to freezing, are uncommon. Usually the situation is that concrete sections tend to dry out during dry seasons, at least one surface being exposed to the atmosphere. That is why air-entrained concrete generally is not damaged by frost action even where nearly all of it is made with absorptive aggregate.

Obviously, the drier the aggregate is at the time the concrete is cast, the more water it must receive to reach critical saturation, and the longer it will take. This is an important consideration, because the length of the wet and cold season is limited. It may prove a disadvantage to use gravel directly from an underwater source, especially if the structure goes into service during the wet season or shortly before the beginning of winter.

Some kinds of rock when dried and then placed in water are able to absorb water rapidly and reach saturation quickly; they are the readily saturable type. This type, even when dry at the start, may reach high levels of saturation while in a concrete mixer, and might not become sufficiently dried by self-desiccation; hence, with such a material, trouble is in prospect if there is not a sufficiently long dry period before the winter season sets in. A small percentage of readily saturable rocks in an aggregate can cause serious damage. Rocks which are difficult to saturate, which are generally coarse grained, are less likely to cause trouble. Obviously, data on the proneness to saturation of each kind of rock in an aggregate could be useful.

Whatever the absorption characteristics of a given aggregate, its rate of absorption in concrete is limited by the rate at which water can pass through its envelope of hardened paste. Because the coefficient of permeability of hardened paste is lower the higher its cement content and the longer it has wet-cured, the rate of absorption of any kind of aggregate can be lowered by reducing the water-cement ratio of the paste and by requiring good curing.

1.3—Ice removal agents

When the practice of removing ice from concrete pavements by means of salt (sodium chloride or calcium chloride) became common some years ago, it was soon learned that these materials caused or accelerated surface disintegration in the form of pitting or scaling. (These chemicals also accelerate the corrosion of reinforcement which can cause the concrete to spall, as described in Chapter 4.)

The mechanism by which deicing agents damage concrete is fairly well understood. It is generally agreed that the action is physical rather than chemical. It involves the development of disruptive osmotic and hydraulic pressures during freezing, principally in the paste, similar to ordinary frost action which is described in Section 1.2. It is, however, more severe.

The concentration of deicer in the concrete plays an important role in the development of these pressures. Verbeck and Klieger[16] showed that scaling of the concrete is greatest at intermediate concentrations (3 to 4 percent). Similar behavior was observed for the four deicers tested: calcium chloride, sodium chloride, urea, and ethyl alcohol. Browne and Cady[17] drew similar conclusions. Litvan's findings[18,19] were consistent with the above mentioned studies. He further concluded that deicing agents cause a high degree of saturation in the concrete, and this is mainly responsible for their detrimental effect. Sale solutions (at a given temperature) have a lower vapor pressure than water; therefore, little or no drying takes

TABLE 1.4.3—RECOMMENDED AIR CONTENTS FOR FROST-RESISTANT CONCRETE

Nominal maximum aggregate size, in. (mm)	Average air content, percent*	
	Severe exposure†	Moderate exposure‡
3/8 (9.5)	7½	6
½ (12.5)	7	5½
¾ (19)	6	5
1½ (38)	5½	4½
3§ (75)	4½	3½
6§ (150)	4	3

*A reasonable tolerance for air content in field construction is ±1½ percent.

†Outdoor exposure in a cold climate where the concrete may be in almost continuous contact with moisture prior to freezing, or where deicing salts are used. Examples are pavements, bridge decks, sidewalks, and water tanks.

‡Outdoor exposure in a cold climate where the concrete will be only occasionally exposed to moisture prior to freezing, and where no deicing salts will be used. Examples are certain exterior walls, beams, girders, and slabs not in direct contact with soil.

§These air contents apply to the whole mix, as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than 1½ in. (38 mm) is removed by handpicking or sieving and the air content is determined on the minus 1½ in. (38 mm) fraction of the mix. (The field tolerance applies to this value.) From this the air content of the whole mix is computed.

There is conflicting opinion on whether air contents lower than those given in the table should be permitted for high strength (more about 5500 psi) (37.8 MPa) concrete. This committee believes that where supporting experience and/or experimental data exists for particular combinations of materials, construction practices, and exposure, the air contents may be reduced by approximately 1 percent. [For maximum aggregate sizes over 1½ in. (38 mm), this reduction applies to the minus 1½ in. (38 mm) fraction of the mix.]

place between wettings (see Section 1.2.3) or cooling.

There have been a few examples of excellent performance of non-air-entrained concrete subjected to deicing salts attributable to a very low water-cement ratio, excellent aggregates, and superior construction practices. Fortunately, however, air entrainment was discovered at about the time the use of deicers became widespread. The benefit from entrained air in concrete exposed to deicers is explained in the same way as for ordinary frost action. Laboratory tests and field experience have confirmed that air entrainment greatly improves resistance to deicers and is actually essential under severe conditions. It is now possible to consistently build scale-resistant pavements.

1.4—Recommendations for durable structures

Concrete which will be exposed to a combination of moisture and cyclic freezing requires the following:

1. Design of the structure to minimize exposure to moisture
2. Low water-cement ratio
3. Air entrainment
4. Suitable materials
5. Adequate curing
6. Special attention to construction practices

These items are described in detail below.

1.4.1 *Exposure to moisture*—Since the vulnerability of concrete to cyclic freezing is influenced so greatly by the concrete's degree of saturation, every precaution should be taken to minimize water uptake. Much can be accomplished along these lines by careful initial design of the structure.

The geometry of the structure should promote good drainage. Tops of walls and all outer surfaces should be sloped. Low spots conducive to the formation of puddles should be avoided. Weep holes should not discharge over the face of exposed concrete. Drainage from higher ground should not flow over the tops or faces of concrete walls.[20]

Unnecessary joints should be eliminated and provisions for suitable drainage should be made. Drip beads can prevent water from running under edges of structural members. "Water traps" or reservoirs, such as may result from extending diaphragms to the bent caps of bridges, should not be designed into the structure.

Even though it is seldom possible to keep moisture from the underside of slabs on grade, subbase foundations incorporating the features recommended in ACI 316-74[21] will minimize moisture buildup. Care should also be taken to minimize structural cracks which may collect or transmit water.

Extensive surveys of concrete bridges and other structures have shown a striking correlation between freezing and thawing damage of certain portions, and excessive exposure to moisture of these portions due to the structural design.[20,22-24]

1.4.2 *Water-cement ratio*—Frost-resistant regular weight concrete should have a water-cement ratio not to exceed the following:

Thin sections (bridge decks, railings, curbs, sills, ledges, and ornamental works) and sections with less than 1 in. (25 mm) of cover over the reinforcement, and any concrete exposed to deicing salts	0.45
All other structures	0.50

Because the determination of absorption of lightweight aggregates is uncertain, it is impracticable to calculate the water-cement ratio of concretes containing such aggregates. For these concretes, a specified 28-day compressive strength of 4000 psi (27.6 MPa) is recommended. For severe exposures, some have found it also desirable to specify a minimum cement content of 564 lb per cu yd (335 kg/m^3), and only that amount of water necessary to achieve the desired consistency.

1.4.3 *Entrained air*—Too little entrained air will not protect cement paste against cyclic freezing. Too much air will unduly penalize the strength. Recommended air contents of concrete are given in Table 1.4.3.

It will be noted that air contents are given for two conditions of exposure—severe and moderate. These values provide about 9 percent of air in the mortar for severe exposure, and about 7 percent for moderate exposure.

Air-entrained concrete is produced through the use of an air-entraining admixture (added at the concrete mixer), an air-entraining cement, or both if necessary.[25] The resulting air content depends on the cement, mix proportions, slump, aggregates, type of mixer, mixing time, temperature, and other factors (including the presence of any material in the mix which increases or decreases the air content). Where an admixture is used, the dosage is varied as necessary to give the desired air content. This is not possible where an air-entraining cement alone is used, and occasionally the air content will be inadequate or excessive. Nevertheless, this is the most convenient method for providing some assurance of protection from cyclic freezing on small jobs where equipment to check the air content is not available. Obviously the preferred procedure is to use air-entraining admixtures.

Frequent determinations of the air content of the concrete should be made. For regular weight concrete, the following test methods may be used: volumetric method (ASTM C 173), pressure method (ASTM C 231), or the unit weight test (ASTM C 138). An air meter may be used to provide an approximate indication of air content. For lightweight concrete, the volumetric method is recommended.

The air content and other characteristics of the air void system in hardened concrete may be determined microscopically (ASTM C 457). ASTM C 672 is often used to assess the resistance of concrete to deicer scaling.

1.4.4 *Materials*

1.4.4.1 *Cementing materials.* The different types of portland and blended cements, when used in properly proportioned and manufactured air-entrained concrete, will provide similar resistance to cyclic freezing. Cement should conform to ASTM C 150 or C 595.

Most pozzolans when used as admixtures have little effect on the frost resistance of concrete provided the air content, strength, and moisture content of the concrete are similar. However, a suitable investigation should be made before using new or questionable materials. Pozzolans should conform to ASTM C 618.

1.4.4.2 *Aggregates.* Natural aggregates should meet the requirements of ASTM C 33, although this will not necessarily assure their durability. Lightweight aggregates should meet the requirements of ASTM C 330. These specifications provide many requirements but leave the final selection of the aggregate largely up to the judgment of the concrete engineer. If the engineer is familiar with the field performance of the aggregate proposed, his judgment may be quite adequate. In some situations it may be possible to carry out field service record studies to arrive at a basis for acceptance or rejection of the aggregate. When this is not feasible, heavy reliance must be placed on laboratory tests.

Laboratory tests on the aggregate include absorption, specific gravity, soundness tests, and determination of the pore structure. Descriptions of the tests, and opinions on their usefulness, have been published.[26,27] Although these data are useful, and some organizations have felt justified in setting test limits on aggregates, it is generally agreed that principal reliance should be placed on tests of concrete made with the aggregate in question.

Petrographic studies of both the aggregate[28] and concrete[29,30] are useful for evaluating the physical and chemical characteristics of the aggregate and concrete made from it.

Laboratory tests on concrete include the rapid freezing and thawing tests (ASTM C 666), where the durability of the concrete is measured by the reduction in dynamic modulus of elasticity of the concrete. Many agencies believe this is the most reliable indicator of the relative durability of an aggregate.

The results of tests using ASTM C 666 have been widely analyzed and discussed.[26,27,31,32] These tests have been criticized because they are accelerated tests and do not duplicate conditions in the field. It has been pointed out that test specimens are initially saturated, while this is not normally the case for field concretes at the beginning of the freezing season. Furthermore, the test methods do not realistically duplicate the actual moisture condition of the aggregates in field concretes. The rapid methods have also been criticized because they require cooling rates greater than those encountered in the field. Also, the small test specimens used are unable to accommodate large aggregate sizes, which may be more vulnerable to deterioration than smaller sizes.

It is rather generally conceded that while these tests may classify aggregates from excellent to poor in approximately the correct order, they are unable to predict whether a fair aggregate will give satisfactory performance when used in concrete with a particular moisture content and cyclic freezing exposure. The ability to make such a determination would be of great economic importance in many areas where high grade aggregates are in short supply, by permitting the use of local marginal aggregates.

Because of these objections to ASTM C 666, a dilation test was conceived by Powers[3] and further developed by others.[33,34] ASTM C 671 requires that air-entrained concrete specimens be initially brought to the moisture condition expected for the concrete at the start of the winter season, this moisure content preferably having been determined by field tests. The specimens are then immersed in water and periodically frozen at the rate and frequency to be expected in the field. The increase in length (dilation) of the specimen during the freezing portion of the cycle is accurately measured. ASTM C 682 assists in interpreting the results.

An excessive length change in this test is an indication that the aggregate has become critically saturated and vulnerable to damage. If the time to reach critical saturation is less than the duration of the freezing season at the job site, the aggregate is judged unsuitable for use in that exposure. If it is more, it is judged that the concrete will not be vulnerable to cyclic freezing.

The time required for conducting dilation tests may be greater than required by other cyclic freezing tests. Also, the test results are very sensitive to the moisture content of the aggregate and concrete. Results are fairly promising,[35] although most agencies are continuing to use ASTM C 666 pending improvements in C 671.

When natural aggregates are found to be unacceptable by service records and/or tests, they may sometimes be improved by removal of lightweight, soft, or otherwise inferior particles in processing.

1.4.4.3 *Admixtures.* Air-entraining admixtures should conform to ASTM C 260.

Chemical admixtures should conform to ASTM C 494. Some such admixtures do not impart adequate durability, even though they entrain sufficient air, because they produce coarse air void systems with void spacing factors greater than the 0.008 in. (0.20 mm) needed to adequately protect concrete. These admixtures should be required to meet ASTM C 260 in addition to any other requirements.

Some mineral admixtures (especially emulsified carbon black, and fly ashes having a high carbon content) will require a larger amount of air-entraining admixture to develop the required amount of entrained air. Dirty aggregates have a similar effect.

Detailed guidance on the use of admixtures is provided by ACI Committee 212.[24]

1.4.5 *Curing*—Air-entrained concrete should be able to withstand one or two freezing and thawing cycles as soon as it attains a compressive strength of about 500 psi (3.45 MPa) provided there is no external source of moisture. At temperatures of 50 F (10 C), most well-proportioned concrete will reach this strength sometime during the second day.

Before being exposed to extended freezing in a severe exposure, it is desirable that concrete attain a specified compressive strength of 4000 psi (27.6 MPa). A period of drying following curing is advisable. For moderate exposure conditions, a specified strength of 3000 psi (20.7 MPa) should be attained.

1.4.6 *Construction practices*—Good construction practices are essential where durable concrete is required.

Particular attention should be given to the construction of pavement slabs to be exposed to deicing salts because of the problems inherent in obtaining durable slab finishes, and the severity of the exposure. The concrete in such slabs should be adequately consolidated; however, overworking the surface, overfinishing, and the addition of water to aid in finishing must be avoided because they bring excessive mortar or water to the surface. The resulting laitance is particularly vulnerable to the action of ice removal salts. These practices may also remove entrained air. This is of little consequence where only the larger air bubbles are expelled, but durability can be seriously affected if the small bubbles are removed.

Prior to the application of any deicer, pavement concrete should have received some drying and the strength level specified for the opening of traffic should have been achieved. These recommendations should be considered in the scheduling of late fall paving. In some cases, it may be possible to employ methods other than ice removal agents, such as abrasives, for control of slipperiness where it is felt the concrete may still be vulnerable.

Where greater than normal protection is needed or for additional insurance, such as for pavements placed in the fall subjected to deicing salts the first winter, a water-repellent surface treatment is advisable (see Section 7.1).

Where lightweight concrete is proposed, care should be exercised not to excessively saturate the aggregate prior to mixing. Saturation by vacuum or thermal means (where necessary for pumping, for example) may bring lightweight aggregates to a condition where the absorbed water may cause concrete failure when cyclically frozen—unless the concrete has the opportunity to dry out before freezing. Additional details and recommendations are given in Reference 35.

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CHAPTER 2—AGGRESSIVE CHEMICAL EXPOSURE

2.1—General

Concrete can be made which will perform satisfactorily when exposed to various atmospheric conditions, to most waters and soils containing chemicals, and to many other kinds of chemicals. There are, however, some chemical environments under which the useful life of even the best concrete will be short. An understanding of these conditions permits measures to be taken to prevent or reduce deterioration.

Concrete is rarely, if ever, attacked by solid, dry chemicals. In order to produce significant attack on concrete, corrosive chemicals must be in solution form and above some minimum concentration. Concrete which is subjected to aggressive solutions under pressure on one side is more vulnerable than otherwise, because the pressures tend to force the aggressive solution into the concrete.

Comprehensive tables have been prepared by ACI Committee 515[1] and the Portland Cement Association[2] giving the effect of many chemicals on concrete. Biczok[3] gives a detailed discussion of the deteriorating effect of chemicals on concrete, including data both from Europe and the United States.

The effects of some of the more common chemicals on the deterioration of concrete are summarized in Table 2.1. It should be kept in mind that there are numerous factors which influence the ability of concrete to resist deterioration, some of which are shown below:

Factors increasing deterioration	*Factors decreasing deterioration*
Higher temperatures	Lower water-cement ratios
Increased fluid velocities	Proper cement type (in some circumstances)
Poor consolidation of concrete	Lower absorption
Poor curing of concrete	Lower permeability
Alternate wetting and drying	
Corrosion of reinforcing steel	

Therefore Table 2.1 should be considered as only a preliminary guide.

Chemical attack on concrete is generally the result of exposure to sulfates or acids, and these are discussed in some detail below.

2.2—Sulfate attack

2.2.1 *Occurrence*—Naturally occurring sulfates of sodium, potassium, calcium, or magnesium are sometimes found in soil or dissolved in groundwater adjacent to concrete structures, and they can attack concrete. When evaporation can take place from an exposed face, the dissolved sulfates (salts) may accumulate at that face, thus increasing their concentration and potential for deterioration. Sulfate attack has occurred at various locations throughout the world, and is a particular problem in arid areas, such as the northern Great Plains area of the United States and the prairie provinces of Canada,[4,5] and in parts of the western United States.[6]

The water used in concrete cooling towers can also be a potential source of sulfate attack because of the gradual build-up of sulfates from evaporation of the water, particularly where such systems use relatively small amounts of make-up water. Sulfates are also present in groundwater in fill containing blast furnace slag or cinders.

2.2.2 *Mechanism*—As Lea,[7] Mehta,[8] and others point out, there are apparently two chemical reactions involved in sulfate attack on concrete.

1. Combination of sulfate with free calcium hydroxide (hydrated lime) liberated during the hydration of the cement, to form calcium sulfate (gypsum).

2. Combination of gypsum and hydrated calcium aluminate to form calcium sulfoaluminate (ettringite).

Both of these reactions result in an increase in solid volume. The latter is generally blamed for most of the expansion and disruption of concretes caused by sulfate solutions.

In addition to the chemical reactions, Tuthill[9] and Reading[5] cite evidence that a purely physical action (not involving the cement), crystallization

TABLE 2.1—EFFECT OF COMMONLY USED CHEMICALS ON CONCRETE

Rate of attack at ambient temperature	Inorganic acids	Organic acids	Alkaline solutions	Salt solutions	Miscellaneous
Rapid	Hydrochloric Hydrofluoric Nitric Sulfuric	Acetic Formic Lactic	—	Aluminum chloride	—
Moderate	Phosphoric	Tannic	Sodium hydroxide— > 20 percent*	Ammonium nitrate Ammonium sulfate Sodium sulfate Magnesium sulfate Calcium sulfate	Bromine (gas) Sulfite liquor
Slow	Carbonic	—	Sodium hydroxide 10-20 percent* Sodium hypochlorite	Ammonium chloride Magnesium chloride Sodium cyanide	Chlorine (gas) Seawater Softwater
Negligible	—	Oxalic Tartaric	Sodium hydroxide < 10 percent* Sodium hypochlorite Ammonium hydroxide	Calcium chloride Sodium chloride Zinc nitrate Sodium chromate	Ammonia (liquid)

*Avoid siliceous aggregates because they are attacked by strong solutions of sodium hydroxide.

of the sulfate salts in the pores of the concrete, can account for considerable damage. Reading also found that where heavy sections are exposed to a strong sulfate solution on the backside, most of the damage is confined to the outer surface adjacent to leaking joints and cracks.

The chemical deterioration of concrete in seawater has concerned concrete technologists for generations, and the discussion continues with respect to the mechanism itself and its practical importance.[7,10] Seawater has a high sulfate content, and it might be expected that stringent measures would be needed to prevent chemical attack. Actually, experience has shown that seawater is only moderately aggressive to concrete. It has been suggested by some that the chlorides in seawater mitigate the action of the sulfates.

2.2.3 *Recommendations*—Protection against sulfate attack is obtained by using a dense, high quality concrete with low water-cement ratio, and a portland cement having the needed sulfate resistance. Air entrainment is of benefit insofar as it reduces the water-cement ratio.[11]

There is fairly good correlation between the sulfate resistance of cement and its tricalcium aluminate (C_3A) content.[12] Accordingly, ASTM C 150 includes a Type V (sulfate resisting) cement which sets a maximum of 5 percent on C_3A, and a Type II (moderately sulfate resisting) cement which limits the C_3A to 8 percent. There is also some evidence that a high C_4AF is detrimental, and for Type V cement the $C_4AF + 2\ C_3A$ must not exceed 20 percent.

In the case of Type V cement, the sulfate expansion test (ASTM C 452) may be used in lieu of the above chemical requirements.

Recommendations for the type of cement and water-cement ratio for normal weight concrete which will be exposed to sulfates in soil, groundwater, or seawater are given in Table 2.2.3. The values also apply to areas in the splash or spray zone.

These values are also applicable to structural lightweight concrete except that the maximum water-cement ratios of 0.50 and 0.45 should be replaced by specified 28-day strengths of 3750 and 4250 psi (25.8 and 29.4 MPa), respectively.

TABLE 2.2.3—RECOMMENDATIONS FOR NORMAL WEIGHT CONCRETE SUBJECT TO SULFATE ATTACK

Exposure	Water soluble sulfate (SO_4) in soil, percent	Sulfate (SO_4) in water, ppm	Cement	Water-cement ratio, maximum*
Mild	0.00-0.10	0-150	—	—
Moderate†	0.10-0.20	150-1500	Type II, IP(MS), IS(MS)	0.50
Severe	0.20-2.00	1500-10,000	Type V	0.45
Very severe	Over 2.00	Over 10,000	Type V + Pozzolan‡	0.45

*A lower water-cement ratio may be necessary to prevent corrosion of embedded items. See Section 4.5.1.1.
†Seawater also falls in this category.
‡Use a pozzolan which has been determined by tests to improve sulfate resistance when used in concrete containing Type V cement.

Studies have shown that some pozzolans used either in blended cement or added separately to the mixer, in the amount of approximately 15 to 25 percent of the portland cement, increase the life expectancy of concrete in sulfate exposures considerably. Pozzolans combine with the free lime resulting from the hydration of the cement, and thereby reduce the amount of gypsum formed.[3,7,8,13] It will be noted that Table 2.2.3 requires a suitable pozzolan along with Type V cement in very severe exposures. Recent research has indicated that pozzolans may be effective in the other categories as well; that is, most Type I cements, even where the C_3A content of the portland cement clinker exceeds the 8 percent allowed for Type IP(MS), would be suitable for moderate exposures and most Type II cements would be suitable for severe exposures if a suitable pozzolan is added. Best results have been obtained when the pozzolan is a fly ash meeting the requirements of ASTM C 618 Class F.[14,15]

The effectiveness of the combination probably depends on the chemical composition of the portland cement and pozzolan, the percentage of each in the mixture, and other factors. It is not yet feasible, however, to predict performance on the basis of these factors. Actual tests of the combination should be made using mildly accelerated test procedures such as those described in Reference 14; information from long-time field performance in structures or field exposure stations should also be considered where available. A highly accelerated test corresponding to ASTM C 452 for portland cement would be very helpful and is now under development in ASTM.

The addition of calcium chloride to concrete reduces its resistance to attack by sulfates,[6] and its use should be prohibited in the last two exposure categories in Table 2.2.3.

It is recognized that these recommendations are conservative, being intended to insure long life construction. Less stringent requirements are permitted by certain agencies where dictated by their experience and shorter life requirements.

2.3—Acid attack

In general, portland cement does not have good resistance to acid attack, although weak acids can be tolerated.

2.3.1 *Occurrence*—The products of combustion of many fuels contain sulfurous gases which combine with moisture to form sulfuric acid. Also, sewage may be collected under conditions which lead to acid formation.

Water draining from some mines, and some industrial waters, may contain or form acids which attack concrete.

Peat soils may contain iron sulfide (pyrite) which, upon oxidation, produces sulfuric acid. Further reaction may produce sulfate salts, which can produce sulfate attack[16]

Mountain water streams are sometimes mildly acidic, due to dissolved free carbon dioxide. Usually these waters attack only the surface if the concrete is of good quality. However, some mineral waters containing large amounts of either dissolved carbon dioxide or hydrogen sulfide, or both, can seriously damage any concrete.[17] In the case of hydrogen sulfide, bacteria that convert this compound to sulfuric acid may play an important role.[7]

Organic acids from farm silage, or from manufacturing or processing industries such as breweries, dairies, canneries, and wood pulp mills, can cause surface damage. This may be of considerable concern in the case of floors, even where their structural integrity is not impaired.

2.3.2 *Mechanism*—The deterioration of concrete by acids is primarily the result of a reaction between these chemicals and the calcium hydroxide of the hydrated portland cement. (Where limestone and dolomitic aggregates are used, they are also subject to attack by acids.) In most cases the chemical reaction results in the formation of water-soluble calcium compounds which are then leached away by the aqueous solutions.[3] Oxalic and phosphoric acid are exceptions, because the resulting calcium salts are insoluble in water and are not readily removed from the concrete surface.

In the case of sulfuric acid attack, additional or accelerated deterioration results because the calcium sulfate formed will affect concrete by the sulfate attack mechanism described in Section 2.2.2.

If acids or salt solutions are able to reach the reinforcing steel through cracks or pores in the concrete, corrosion of the steel can result (see Chapter 4) which will in turn cause cracking and spalling of the concrete.

2.3.3 *Recommendations*—A dense concrete with low-water cement ratio may provide an acceptable degree of protection against mild acid attack.

No portland cement concrete, regardless of its composition, will long withstand water of high acid concentration. In such cases, an appropriate surface coating or treatment must be used. The ACI Committee 515 report[1] gives recommendations for barrier coatings to protect concrete from various chemicals. Chapter 7 of this guide discusses the general principles involved in the use of coatings.

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CHAPTER 3—ABRASION

3.1—Introduction

The abrasion resistance of concrete is defined as the "ability of a surface to resist being worn away by rubbing and friction."[1] Abrasion of floors and pavements may result from production operations, or foot or vehicular traffic; abrasion resistance is therefore of concern in industrial floors.[2] Wind or waterborne particles can also abrade concrete surfaces.[3] There are instances where abrasion is of little concern structurally, yet there may be a dusting problem which can be quite objectionable in some kinds of service. Abrasion (erosion) of concrete in hydraulic structures is discussed only briefly in this guide; the subject is treated in detail in the report of ACI Committee 210.[4]

3.2—Testing concrete for resistance to abrasion

Research to develop meaningful laboratory tests on concrete abrasion has been underway for more than a century. The problem is complicated because there are several different types of abrasion, and no single test method has been found which is adequate for all conditions. Following the recommendations of Prior,[5] abrasion is classified into four types:

1. Wear on concrete floors due to foot traffic and light trucking, skidding, scraping, or sliding of objects on the surface (attrition)

2. Wear on concrete road surfaces due to heavy trucks and automobiles with studded tires or chains (attrition, scraping and percussion)

3. Erosion in hydraulic structures such as dams, spillways, tunnels, and bridge abutments, due to the action of abrasive materials carried by flowing water (attrition and scraping)

4. Wear on concrete dams, spillways, tunnels, and other water-carrying systems where high velocities and negative pressures are present. This is generally known as cavitation erosion which is mainly the result of design and is not covered in this guide.

In 1974 ASTM adopted C 779-74, "Standard Method of Test for Abrasion Resistance of Horizontal Concrete Surfaces." It includes three optional procedures: (1) the revolving-disc adaptation of the Schuman and Tucker machine,[6] (2) the dressing-wheel machine, and (3) the ball-bearing machine.

In order to properly evaluate abrasion resistance, the type of concrete being tested must be considered. If it is of the same mix throughout, the abrasion resistance can be expected to be a direct function of the concrete strength. If, however, metallic or other hardeners have been applied, the time required for the abrasion apparatus to penetrate the hard surface must be determined to properly evaluate the test results.

It is not yet possible to set precise limits for abrasion resistance of concrete. It is instead necessary to rely on relative values based on weight or volume loss, depth of wear, or visual inspection for judging the wearing qualities of concrete surfaces. This is not to suggest that the laboratory tests are not useful, but rather that field experience provides more reliable data.

3.3—Factors affecting abrasion resistance of concrete

The abrasion resistance of concrete is affected primarily by:

1. Compressive strength
2. Aggregate properties
3. Finishing methods
4. Use of toppings
5. Curing

Tests[7,8] and field experience have generally shown that compressive strength is the most important single factor controlling the abrasion resistance of concrete, with abrasion resistance increasing with increase in compressive strength. Compressive strength and abrasion resistance vary inversely with the ratio of voids (water plus air) to cement. For rich mixes, limiting the maximum size of the aggregate will improve compressive strengths and result in maximum abrasion resistance of concrete surfaces.

Another highly important element is the abrasion resistance[7,9] of the coarse and fine aggregate (at the surface). The service life of some concrete, such as warehouse floors subjected to abrasion by steel or hard rubber-wheeled traffic, may be greatly lengthened by the use of especially hard and tough aggregates. The effect of differences in hardness between aggregates is more pronounced in lower strength concrete; it becomes less in high strength concrete and toppings.

Proper finishing procedures and timing are essential if the quality of concrete near the surface of a slab is to be as good as that for the underlying section. Delaying the floating and troweling operations increases resistance to abrasion.

In a two-course floor it is possible to provide a surface course having excellent abrasion resistance, while using ordinary concrete in the remainder of the slab.

Applying to the surface dry shake coats of cement and hard fine aggregate, or of cement and iron aggregate, will also make the surface layer more abrasion resistant.

Another highly important ingredient in wear-resistant, nondusting concrete surfaces is adequate curing.[5,10,11] One study showed that a surface cured for 7 days is nearly twice as wear-resistant as one cured for only 3 days, and additional curing resulted in further improvement.[10].

In cold weather concreting, carbon dioxide from unvented heaters can have a detrimental effect on abrasion resistance.

3.4—Recommendations for obtaining abrasion-resistant concrete surfaces

The following measures will result in abrasion-resistant concrete surfaces.

3.4.1 *Appropriate concrete strength level*—The strength selected should be appropriate for the service and time. In no case should the compressive strength be less than 4000 psi (27.6 MPa). Suitable strength levels may be attained by:

1. A low water-cement ratio

2. Proper grading of fine and coarse aggregate (meeting ASTM C 33). Limit the maximum size to nominal 1 in. (25 mm)

3. Lowest consistency practicable for proper placing and consolidation. Maximum slump of 3 in. (75 mm), 1 in. (25 mm) for toppings

4. Minimum air content consistent with the exposure conditions. For indoor floors not subjected to freezing and thawing, air contents of 3 percent or less are preferable. In addition to a detrimental effect on strength, high air contents

can cause blistering—particularly when using dry shakes.

3.4.2 *Two-course floors*—When wear conditions are severe, use a high strength [not less than 5000 psi (34.5 MPa)] topping layer.[10] Limit the maximum size of aggregate to ½ in. (12 mm) in the topping.

3.4.3 *Special aggregates*—Using hard, tough coarse aggregates will provide additional abrasion resistance. In two-course floors, they need be included in only the topping layer.

3.4.4 *Proper finishing procedures*—Delay floating and troweling until the concrete has lost its surface water sheen or all free water on the surface has disappeared or been carefully removed. The delay period is usually for 2 or more hr after placing the concrete (depending on temperatures, mix proportions, and air content). Follow the recommendations of ACI 302-69[10] and 304-73[12] with respect to finishing of unformed surfaces.

3.4.5 *Vacuum dewatering*—Vacuum dewatering is a method of removing excess water from concrete immediately after placement. The process results in increased strength, hardness, and wear resistance of concrete surfaces; it is primarily applicable to slab.[13] ACI Committee 302 is investigating this procedure.

3.4.6 *Special dry shakes and toppings*—Where severe wear is anticipated, the use of special toppings or dry shakes should be considered and, if selected, the recommendations of ACI Committee 302 should be followed.

3.4.7 *Proper curing procedures*—Curing should start immediately after the concrete has been finished and be continued for at least 7 days with Type I cement (5 days with Type III). Curing with water by spray, damp burlap, or cotton mats is preferred, provided the concrete is kept continuously moist. Waterproof paper or plastic sheets are satisfactory, provided the concrete is first sprayed with water and then immediately covered with the paper or plastic with the edges overlapped and sealed with waterproof tape. Curing compounds meeting ASTM C 309 seal the moisture in the concrete and are economical and easy to apply; they may be used where other methods are impracticable. The curing compound should be covered with scuff-proof paper if a floor area must be used before curing is completed. Curing compounds must be specially formulated where paint or resilient tile are to be applied later on. Curing methods are described in detail in ACI 308-71.[11]

Unvented salamanders or other unvented heaters producing carbon dioxide should not be used during cold weather floor construction because they produce carbon dioxide gas.[14] Nor should gasoline-powered equipment be used in enclosed areas because the carbon monoxide fumes are dangerous to human life and also can damage floor surfaces.

3.5—Improving wear resistance of existing floors

Liquid surface treatments (hardeners) may sometimes be used to improve the wear resistance of floors.[15] Magnesium or zinc fluosilicate, or sodium silicate, are most commonly used. Their principal benefit is in reducing dusting. They may also slightly resist deterioration by some oils and chemicals coming in contact with the concrete. Liquid hardeners are most useful on older floors which have started to abrade or dust, as a result of poor quality concrete or poor construction practices (particularly finishing while bleed water is on the surface, and inadequate curing). In such cases, they serve a useful purpose in prolonging the service life of the floor. New floors should be of such quality that treatments with liquid hardeners should not be required, except where even slight dusting cannot be tolerated (as in powerhouse floors).

Liquid hardeners should not be applied to new floors until they are 28 days old. The floor should be moist cured and then allowed to air dry during this period. Curing compounds should not be used if hardeners are to be applied, because they reduce the penetration of the liquid into the concrete. The hardener should be applied in accordance with the manufacturer's instructions.

3.6—Studded tire wear on concrete pavements

Tire chains and studded snow tires cause considerable wear to concrete surfaces, even where the concrete is of good quality. Abrasive materials such as sand are often applied to the pavement surface when roads are slippery. However, experience from many years' use of sand in winter indicates that this causes little wear if the concrete is of good quality and the aggregates are wear-resistant.

In the case of tire chains, wear is caused by a flailing and scuffing action as the rotating tire brings the metal in contact with the concrete surface. Fortunately, the use of chains is limited mainly to roads in the snow belt or mountain areas, and even there they are used only when essential.

Studded snow tires have caused widespread and serious damage, even to high quality concrete. In this case the damage is due to the dynamic impact of the small tungsten carbide tip of the studs, of which there are roughly 100 in each tire. Pavement surfaces in the northern United States, Canada, and the northern European countries, where pavements are bare for much of the winter season and these tires remain on until the spring, have suffered the most. One laboratory study showed that studded tires running on surfaces to which sand and salt were applied caused 100 times as much wear as unstudded tires.[16]

Wear caused by studded tires is usually concentrated in the wheel tracks. Ruts from ¼ to ½ in. (6 to 12 mm) deep may form in a single winter in regions where approximately 30 percent of passenger cars are equipped with studded tires and traffic is heavy.[17] More severe wear occurs where vehicles stop, start, or turn.[17,18]

Investigations have been made, principally in Scandinavia, Canada, and the United States, to examine the properties of existing concretes as related to studded tire wear.[17-23] In some cases there was considerable variability in the data, and the conclusions of the different investigators were not in agreement. However, most found that a hard coarse aggregate and high strength mortar matrix are somewhat beneficial in resisting abrasion.

Another investigation in the United States was aimed at developing more wear-resistant types of concrete overlays.[24] It was concluded that polymer cement and polymer fly ash concretes provided better resistance to wear—though at the sacrifice of skid resistance. Steel fibrous concrete overlays were also tested and showed reduced wear as compared with sections of regular concrete. Although these results are fairly promising, no "affordable" concrete surface has yet been developed which will provide a wear life when studded tires are used approaching that of normal surfaces under rubber tire wear.

A recently published report[25] summarizes available data on pavement wear, and on the performance and winter accident record while studded tires have been in use.

3.7—Skid resistance of pavements

The skid resistance of concrete pavement depends on its surface texture. Two types of texture are involved:

1. Macro (large scale) texture resulting from surface irregularities "built in" at the time of construction

2. Micro (small scale) texture resulting from the harshness and type of fine aggregate used

The micro texture is the more important, particularly at speeds of less than about 50 mph (80 km/hr).[26-28]

The skid resistance of concrete pavement initially depends on the texture built into the surface layer. In time, rubber-tired traffic abrades the immediate surface layer, removing the beneficial macro texture and eventually exposing the coarse aggregate particles. The rate at which this will occur and the consequences on the skid resistance of the pavement depend on the depth and quality of the surface layer and the rock types in the fine and coarse aggregate.

Fine aggregates containing significant amounts of silica in the larger particle sizes will assist in slowing down the rate of wear and maintaining the micro texture necessary for satisfactory skid resistance at the lower speeds. Certain rock types, however, polish under rubber tire wear. These include very fine textured limestones, dolomites, and serpentine. Where both the fine and coarse aggregate are of this type there may be a rapid polishing of the entire pavement surface with a serious reduction in skid resistance. Where only the coarse aggregate is of the polishing type, the problem is delayed until the coarse aggregate is exposed by wear. On the other hand, if the coarse aggregate is, for example, a coarse grained silica or vesicular slag, the skid resistance may be increased when it is exposed.

At speeds greater than about 50 mph (80 km/hr), the macro texture becomes quite important because it must be relied on to prevent hydroplaning. This texture is accomplished by constructing grooves in the concrete—either during the plastic stage or by sawing later on—to provide channels for the escape of water otherwise trapped between the tire and pavement. It is vital that the "islands" between the grooves be particularly resistant to abrasion and frost action. A high quality concrete, properly finished and cured, possesses the required durability.

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CHAPTER 4—CORROSION OF STEEL AND OTHER MATERIALS EMBEDDED IN CONCRETE

4.1—Introduction

Under most conditions, portland cement concrete provides adequate protection of embedded materials against corrosion, (i.e., rusting, oxidation, etc.). The protective value of the concrete is attributable to its high alkalinity and relatively high electrical resistivity in atmospheric exposure. The degree to which concrete will provide satisfactory protection is in most instances a function of the quality of the concrete, the depth of con- tices are followed throughout the entire construction operation.

Notwithstanding the immunity against corrosion usually afforded by concrete, a disconcerting number of cases have been reported in which corrosion of embedded items has necessitated large expenditures for repair and maintenance. Deterioration of concrete and reinforcement caused by steel corrosion have been reported from many parts of the world.[1-6] Similar problems with

It should be noted that corrosion, or other distress of embedded items, is not only damaging to the item involved but, in most cases, is also damaging to the concrete.

The corrosion of metals is being studied in considerable detail and depth by ACI Committee 222. The purpose of this chapter is to briefly describe corrosion problems encountered with various materials, and to recommend procedures which will provide the required protection (1) under normal circumstances and (2) under unusually severe conditions.

The literature on the subject, with respect to corrosion of steel and to a lesser extent for other embedded materials, is very extensive. It is not practicable to discuss all aspects of corrosion in this chapter or to even cite all of the pertinent references. The references have been selected primarily to provide as broad a coverage as possible and still keep the list within reasonable limits.

4.2—Effect of concrete condition on corrosion of embedded steel

Concrete may not provide permanent protection to embedded items under the following conditions:

Presence of cracks—The common belief that cracks are necessary for widespread damage due to corrosion is erroneous. Corrosion of reinforcing steel can occur in uncracked concrete when the exposure conditions are severe and when the concrete cover over the steel is insufficient. (Adequate concrete cover over the steel for differing exposure conditions is discussed in Section 4.5.2.) Nevertheless, cracks extending in from the concrete surface contribute to corrosion because they may give access to moisture, air, and contaminants. Narrow cracks in a direction transverse to that of the embedded item may not lead to serious corrosion except in severe environments, since the corrosion will probably be localized and shallow. Wider cracks, and particularly cracks parallel to the direction of the embedded item, may provide greater access to corroding substances and thus accelerate their attack while aggravating other causes of corrosion which are discussed later.

Carbonation—Hydrated portland cement is subject to chemical reaction with the carbon dioxide of the atmosphere. Such carbonation increases the shrinkage of concrete on drying and thus tends to promote the development of cracks. Carbonation also reduces the alkalinity of concrete and thus reduces its effectiveness as a protecting medium. In good quality concrete, properly consolidated and cured, carbonation does not penetrate deeply. It is only in the permeable or porous concrete, or where the reinforcement is relatively close to the surface, that corrosion is likely to become a problem from this cause.

Deterioration of concrete from other causes—Where concrete has deteriorated from freeze-thaw cycles, sulfate attack, reactive aggregates, or other cause, it will be cracked or weakened and thus become less able to protect embedded items. Recommendations for making durable concrete are given elsewhere in this report. The point here is that nondurable concrete can contribute to corrosion of embedded materials.

4.3—Causes of corrosion

4.3.1 *Stray electrical currents*—The passage of direct electrical current through concrete or concrete reinforcement may cause rapid and serious corrosion. Such current frequently has been caused by electrical leakage or by failure to provide positive and permanent means of grounding electrical systems. Corrosion from this source should be considered a possibility in the vicinity of any direct current equipment, including electrified railways and cathodically protected pipelines, particularly where there is an electrolyte such as sodium or calcium chloride solution within or in contact with the concrete.

4.3.2 *Corrosion cells within the concrete*—The most frequent form of corrosion of metals in concrete is caused by a flow of electric current generated within the concrete itself. (This is a different source of corrosion than "stray current corrosion," where the current originates externally.) Electrical potential differences can occur in various spots in concrete containing metals because of differences in moisture content, oxygen concentration, electrolyte concentration, and by contact of dissimilar metals. In such concrete, a corrosion cell is set up along a reinforcing bar or other embedded metal through the formation of an anode—where corrosion occurs, and a cathode—which does not corrode. The distance between these parts of a cell can vary from a fraction of an inch to 20 ft or more (from about 10 mm to 6 m or more).

The presence of an electrolyte is necessary for the creation of a corrosion cell. An electrolyte is a solution capable of conducting electric current by ionic flow. Any moist concrete contains enough electrolyte to conduct a corrosion current. The drier the concrete, the lower its conductivity. Exposure of concrete to wetting by water containing soluble salts such as seawater, sulfate bearing waters, chlorides applied to pavements and bridge decks for ice control, and brine drippings from railroad refrigerator cars are sources of danger. In addition, excessive amounts of salts may be incorporated in the fresh concrete by the use of mixing water containing soluble salts (e.g., seawater), aggregates which contain salts (either naturally or through contamination), or by the use of admixtures containing chloride salts.[7]

Another ingredient essential to corrosion of most metals in concrete is oxygen. Therefore, the rate of corrosion (and thus the rate of damage) can be reduced or inhibited by choking off the access to oxygen. Various procedures, one being deep polymer impregnation of chloride contaminated concrete, are being tested for effectiveness in this regard.

Corrosion cells can produce pits in normal reinforcing steel and in some other metals. Loss of section due to pitting or widespread corrosion is of much greater consequence in prestressing cables and other small stressed member than in large reinforcing bars. A catastrophic failure can occur in a stressed cable if the cross section is reduced sufficiently by corrosion or by embrittlement of the steel from hydrogen evolution during the corrosion process. Conversely, the concrete surrounding a large reinforcing bar is often cracked as the result of the expansive forces of corrosion long before loss of section becomes critical from a load carrying standpoint. In this instance, repairs are often necessary because of loss of bond, cracks, or spalling. Thus, corrosion in either instance is costly.

4.4—Corrosion characteristics of various materials in concrete

4.4.1 *Steel*—In the early stages of corrosion, rust stains (generally dark gray or reddish in color) may be observed in the pores of the concrete and in small cracks at the surface. Later, there is more prominent cracking of the concrete in a direction parallel to the reinforcement, and a delamination (disbonding) of the concrete at the level of the steel. In advanced cases, spalling down to the level of the reinforcement occurs.

Techniques are available for locating areas of corroding steel and subsurface delaminations, and for determining the chloride content (and thus the potential for corrosion of embedded metal) of the concrete at the level of the embedded metal.[8-10] The latter procedure requires that samples of concrete be obtained, whereas the other procedures are nondestructive.

4.4.2 *Aluminum*—Corrosion of aluminum embedded in concrete can occur and can crack the concrete. Conditions conducive to corrosion are created if: the concrete contains steel in contact with the aluminum, chlorides are present in appreciable concentrations, or the cement is high in alkali content.[6] Increasing ratios of steel area to aluminum area (when the metals are coupled) particularly in the presence of appreciable amounts of chloride, increases corrosion of the aluminum. Additionally, hydrogen gas evolution may occur when fresh concrete contacts aluminum and this may increase the porosity of the concrete and therefore the penetration of future corrosive agents. Some aluminum alloys are more susceptible to this problem than others.

4.4.3 *Lead*—Lead in damp concrete can be attacked by the calcium hydroxide in the concrete and may be destroyed in a few years. Contact of the lead with reinforcing steel can accelerate the attack. It is recommended that a protective coating of bituminous material, plastic, or sleeves which are unaffected by damp concrete be used on lead to be embedded in concrete. Corrosion of embedded lead is not likely to damage the concrete.

4.4.4 *Copper and copper alloys*—Copper is not normally corroded by concrete as evidenced by the widespread and successful use of copper waterstops and the embedment of copper pipes in concrete for many years. However, corrosion of copper pipes has been reported where ammonia is present. Also, there have been reports that small amounts of ammonia and possibly of nitrates can cause stress corrosion cracking of embedded copper. It should further be noted that unfavorable circumstances are created if the concrete also contains steel connected to the copper. In this case it is the steel which will corrode.

4.4.5 *Zinc*—Zinc reacts with alkaline materials such as those found in concrete. However, zinc in the form of a galvanizing coating on reinforcing steel is sometimes intentionally embedded in concrete. Available data are conflicting as to the benefit, if any, of this coating.[11-14] A chromate dip on the galvanized bars or the use of 400 parts per million of chromate in the mix water is recommended to prevent hydrogen evolution in the fresh concrete. Additionally, users are cautioned against permitting galvanized and black steel to come in contact with each other in a structure, since theory indicates that the use of dissimilar metals can cause galvanic corrosion.

Some difficulty has been experienced with the corrosion and perforation of corrugated galvanized sheets used as permanent bottom forms for concrete roofs and bridge decks. Such damage has been confined largely to concrete containing appreciable amounts of chloride and to areas where chloride solutions are permitted to drain directly onto the galvanized sheet.

4.4.6 *Other metals*—Chromium and nickel alloyed metals generally have good resistance to corrosion in concrete, as do silver and tin. However, the corrosion resistance of some of these metals may be adversely affected by the presence of soluble chlorides in seawater or deicing salts. Special circumstances might justify the cost of Monel or Type 316 stainless steel in marine location, if data are available to document their superior performance in concrete containing moisture and chlorides or other electrolytes. However, the 300 series stainless steels are susceptible to stress corrosion cracking when the temperature is over 140 F (60 C) and chloride solutions are in contact with the material. Natural weathering steels generally do not perform well in a concrete containing moisture and chloride.

4.4.7 *Plastics*—Plastics are being used increasingly in concrete as pipes, shields, waterstops, chairs, etc., as well as a component in the concrete. Many plastics are resistant to strong alkalies and therefore would be expected to perform satisfactorily in concrete. However, because of the great varieties of plastics and materials compounded with them, specific test data should be developed for each intended use. Special epoxies have been used successfully as reinforcing bar coatings and will be discussed later in this guide. Another rapidly expanding field involving the use of plastics includes polymer impregnated concrete (hardened concrete impregnated with plastic), polymer concrete (where the plastic is the binder) and polymer-portland cement concrete (where the polymer is an additional component of a conventional concrete).[15] The use of polymers can result in a superior concrete, both from the strength and permeability standpoints. Several applications are discussed later on in this report.

4.4.8 *Wood*—Wood has been widely used in or against mortars and concretes. Such use varies from the incorporation of sawdust, wood pulp, and wood fibers in the concrete mix to the embedment of timber.

The incorporation of untreated sawdust, wood chips or fibers in "nailing concrete," and the like, usually results in slow setting and low strength. The addition of hydrated lime equal to ⅓ to ½ the volume of cement has often been effective in overcoming this action. Further improvement has resulted from the use of up to 5 percent of calcium chloride dihydrate along with the lime. However, calcium chloride in such amounts can cause corrosion of embedded metals and can have adverse effects on the concrete itself.

Another problem with such concrete is the high volume change, which occurs even with changes in atmospheric humidity. This volume change may lead to cracking and warping.

The embedment of lumber in concrete has sometimes resulted in leaching of the wood by calcium hydroxide with subsequent deterioration. Softwoods, preferably with a high resinous content, are reported to be most suitable for such use.

4.5—Recommendations where corrosion may be a problem

4.5.1 *Concrete of low permeability*—The permeability of concrete is a major factor affecting the process of corrosion of embedded materials. It is also a major factor affecting the service life of the concrete itself. Less water can enter and remain in a low permeability concrete under a given exposure and hence such concrete is more likely to have low electrical conductivity. It also resists the absorption of salts and their penetration to the embedded items and provides a barrier against the ingress of oxygen. Although no conventional concrete is completely impermeable, proper attention to mix proportioning, workmanship, and curing will give a concrete having a low permeability.

4.5.1.1 *Mix proportioning.* Low water-cement ratios produce less permeable concrete and thus provide greater assurance against corrosion. In seawater exposure tests of reinforced concrete piles with a nominal cover of 1½ in. (38 mm) over the steel, a water-cement ratio of 0.45 (by weight) provided good protection against corrosion, 0.53 provided an intermediate degree of protection, and 0.62 afforded relatively poor protection.[3] Exposure tests on 20 ft^2 (1.85 m^2) slabs which were salted daily yielded similar results; concrete with a water-cement ratio of 0.40 performed significantly better than concretes with water-cement ratios of 0.50 and 0.60, even with equal cement contents.[9]

Therefore, the water-cement ratio should not exceed 0.40 for concrete exposed to sea or brackish water, or in contact with more than moderate concentrations of chlorides at the water or ground line or within the range of fluctuating water level or spray. If this water-cement ratio cannot be achieved in a specific instance, a maximum water-cement ratio of 0.45 may be used provided the thickness of concrete cover over any metal is increased by 0.5 in. (1.3 cm).

Above the sea and spray range for a height of 25 ft (8 m) or within a horizontal distance of 100 ft (30 m), the water-cement ratio should not exceed 0.50 by weight.

These recommended limitations on water-cement ratio apply to all types of portland cement, although long-term studies at PCA on durability of concrete (seawater exposure) showed that cement containing 5 to 8 percent tricalcium aluminate (C_3A) showed less cracking due to steel corrosion than cement with a C_3A content less than 5 percent.[16] In the absence of specific test data, ACI

211.1-74 may be used to determine the cement factor required for the stated water-cement ratio.

A low water-cement ratio does not of itself assure a low permeability concrete. As an extreme example, so-called "no-fines" concrete could have a low water-cement ratio and yet be highly permeable, as evidenced by the use of such concrete to produce porous pipe. Well-graded coarse and fine aggregates are therefore also requisite to low permeability.

Air entrainment is recommended to reduce damage from freezing and thawing and may improve workability. Also, tests have shown that the time for corrosion-caused cracking to develop is increased significantly by incorporating air in the mix.[13]

4.5.1.2 *Workmanship*. Good workmanship is a most important factor in securing uniform concrete of low permeability. This includes the use of low slump concrete, precautions against segregation, thorough vibration to insure good consolidation, and good finishing practices. Low slump concretes are often difficult to consolidate and a density monitoring device (such as a direct transmission nuclear gage) may be helpful.

4.5.1.3 *Curing*. Permeability is reduced by increased hydration of the cement. Therefore, adequate curing is essential. At least 7 days of uninterrupted moist curing, or membrane curing, should be required. Members that are cured with low pressure steam to obtain a high early strength will benefit significantly from additional moist curing at normal temperatures.

4.5.2 *Adequate steel cover*—Protection against penetration of salts to reinforcing steel and other embedded items is affected considerably by the thickness of concrete cover over the steel. It is generally recognized that at or near the waterline or in other locations exposed to a combination of seawater (including spray) and atmospheric (free) oxygen in marine construction and other severe environments, more cover is required than is normally used. A minimum cover of 3 in. (75 mm) is recommended for such exposure [AASHTO recommends 4 in. (100 mm) except for precast piles]. Exposure of concrete at inland sites, other than brackish water, has not generally been recognized as constituting a corrosion problem except for bridge decks. On bridges, salts applied in ice control operations are absorbed by the concrete roadway decks and adjacent appurtenances such as curbs, sidewalks, and railings. In such locations a minimum cover of 2 in. (50 mm) and a concrete of low water-cement ratio (0.40 maximum by weight) are recommended. It should be noted that because of construction tolerances in steel cover on bridge decks, a design or "plan" cover of at least 2.6 in. (65 mm) would need to be specified to obtain a minimum cover of 2.0 in. (50 mm) over 90 to 95 percent of the deck reinforcing steel.[17] A nondestructive magnetic device (pachometer) is available for use in determining the depth of cover over reinforcing steel in hardened concrete.[10,17]

4.5.3 *Good drainage*—In areas of severe exposure, especially in concrete bridge decks, particular attention shoould be given to design details dealing with drainage. They should insure that the water will drain, and standing pools are avoided.

4.5.4 *Limiting chlorides in the concrete mix*—The potential hazard of chlorides to concrete containing steel in a marine environment or other exposure to soluble salts suggests a recommendation that no chloride should be allowed in the concrete mix. This would reject the use of seawater as mixing water, aggregates which have been washed with seawater or otherwise contain salts, and admixtures containing chloride.

Specifying a zero chloride content for the mix, however, is impossible to realize in practice. Chlorides are among the more abundant materials on earth, and are present in variable amounts in all of the ingredients of concrete. Neither is it effective to place a prohibition only on calcium chloride, since other chlorides can react in the same manner. The proper approach is to limit the total chloride in the mix (i.e., in the aggregate, cement, mixing water, and admixtures) to a value less than that required to promote corrosion.

Research has shown that the threshold value for a chloride content in concrete necessary for the corrosion of embedded steel can be as low as 0.15 percent by weight of cement.[9,18-21] At first sight, therefore, this limit should be universally specified. However, this approach is undesirable because it does not take into account the physical availability of the chloride nor does it consider if the other components necessary for corrosion of steel, oxygen and moisture, will be present or not. The availability of oxygen and moisture adjacent to the steel will vary with service exposure from one structure to another and between different parts of the same structure as well as with the quality of concrete and depth of cover to steel provided. Also, prior to any meaningful discussion of limits the form in which the chloride occurs must be taken into account.

Chloride in concrete may be in the water soluble form or may be chemically combined with other ingredients. Soluble chlorides induce corrosion, while combined chloride is believed to have little effect. It has been shown that when the total chloride content is near the corrosion threshold level, from 50 to 85 percent of it will be soluble.[9,19]

There are exceptions, however. Some data indicates that a large percentage of calcium chloride admixture combines chemically with the cement and thus is not available to induce corrosion.[25]* Some chloride-bearing aggregates have a high total chloride content but very little of it is soluble. On the other hand, some aggregates with high chloride content are known to have caused corrosion.

When considering the probability of corrosion, it is therefore logical to measure only the soluble chloride content of concrete, rather than total chloride. Tests for soluble chloride, however, are time-consuming and difficult to control. Factors such as sample size, boiling and/or soaking time, temperature, and quantity of distilled water used all affect the results.[19] Therefore the test must be performed in a standardized manner. Conversely, the test for total chloride, which involves a nitric acid extraction, is not significantly affected by the above factors.[10,21-24] Most interested parties, therefore, measure total (soluble plus combined) chloride and test for soluble chloride only when follow-up studies are desired.

If the total chloride is less than the allowable limit, obviously soluble chloride need not be measured. Should the total chloride content exceed this limit, additional information on the risk involved in using the material may be obtained by performing a soluble chloride test. When this value is found to be above the limit, corrosion is likely if moisture and oxygen are readily available (in a bridge deck, for example). If it is below the limit, the risk of corrosion is low.

At the present state of knowledge this committee, in agreement with ACI Committee 222, suggests the following limits for chloride ion (Cl—) in concrete prior to service exposure, expressed as a percent by weight of cement:

1. Prestressed concrete .... 0.06 percent
2. Conventionally reinforced concrete in a moist environment and exposed to chloride .... 0.10 percent
3. Conventionally reinforced concrete in a moist environment but not exposed to chloride (includes locations where the concrete will be occasionally wetted—such as kitchens, parking garages, waterfront structures, and areas with potential moisture condensation .... 0.15 percent
4. Above ground building construction where the concrete will stay dry .... No limit for corrosion†

The user should exercise good judgment in applying these limits, keeping in mind that other factors (moisture and oxygen) are always necessary for electrochemical corrosion.

The routine measurement of total chloride[24] for direct comparison with the suggested limits is recommended. This may be made on the constituents of the concrete, in concrete from trial batches, or on production concrete shortly after mixing. However, as discussed above, if the total chloride content exceeds the limit, the potential for corrosion can be further studied using the soluble chloride test described in Reference 24. If these results are less than the limit, the probability of corrosion caused by chloride contained in the concrete mix will be low.

No calcium chloride should be intentionally added to the mix in prestressed concrete or conventionally reinforced concrete which will be exposed to moisture and chlorides in service, even if the naturally occurring chlorides in the materials are less than the stated limits.

Obviously the effect of chlorides in concrete is complex. ACI Committee 222 is continuing to study the subject. The recommendations of Committees 212 and 443 are also pertinent.

4.5.5 *Careful attention to protruding items*—When embedded items, such as bolts, must protrude from the concrete in a corrosive environment, careful attention should be given to the resistance of the material selected, to the type of corrosive environment, to the avoidance of coupling it with a dissimilar metal inside the concrete, to the careful placement of the concrete around the protruding item, and to the avoidance of creating channels which will permit the corrosive media to reach the interior of the concrete.

4.5.6 *Positive protective systems*—Because of the very high cost of repairing corrosion-caused damage, positive protective systems are being used for bridge decks in severe deicing salt areas and for some marine structures. Many protective systems have been proposed, some of which have been shown to be effective while others have failed. It is beyond the scope of this guide to discuss all possible systems. However, the most successful systems in use for bridge decks (some of which are applicable to other structures) are listed below:

1. Very low water-cement ratio (0.32 by weight), low slump dense concrete overlay.[9,26]

*This has led some to conclude that up to 1 percent of admixture in the usual flake form (the dihydrate, $CaCl_2 \cdot 2H_2O$) may be acceptable in most conventionally reinforced concrete which does not contain other embedded metals and which will not be exposed to chloride in service.

†If calcium chloride is used as an admixture, a limit of 2 percent is generally recommended for reasons other than corrosion. Using 2 percent of the usual form (the dihydrate, $CaCl_2 \cdot 2H_2O$) results in approximately 1 percent chloride ion (Cl—).

2. Styrene-butadiene latex modified concrete overlay.[9,26]

3. Epoxy (electrostatically applied powder) coated reinforcing steel.[27,28]

4. Specific waterproof membranes with an asphaltic concrete wearing surface.[26,29]

The reader is referred to the indicated references for details.

Other promising protective systems in the final development stages include surface polymer impregnated concrete[30] and internally sealed concrete.[31,32] Galvanized reinforcing steel is being used on an experimental basis; available data on laboratory and field performance are conflicting and generally not encouraging.[12-15,28]

4.6—Corrective measures

Methods of repairing concrete which has deteriorated as a result of corrosion of the reinforcing steel or other embedded items are given in the literature.[33,34] Chapter 6 of this guide also describes repair procedures.

In situations where electrochemical phenomena exist it should be noted that repairs to one part of a structural unit may aggravate corrosion in other parts. Continued repair may well be the most economical solution, but the simple repair of affected portions may not necessarily result in a permanent cure.

Several methods of stopping or retarding further corrosion have been studied and are discussed in Section 4.7.

4.7—General remarks

As stated at the beginning of this chapter, under most conditions, portland cement concrete provides adequate protection of embedded materials against corrosion. Corrosion sometimes occurs, however, and when it does, it can be a very costly experience. The complexity of the causes, high cost of present repairs, and the experimental status of the more promising corrective measures emphasize the importance of good design, the use of good quality concrete, good workmanship, and good curing for the original construction.

In some environments (such as bridge decks and other exposed members in marine and deicing salt environments), the use of a positive protective system at the time of construction is recommended.

It is a difficult and uncertain undertaking to stop corrosion after conditions leading to its development have been built into the structure. Cathodic protection of corroding bridge decks is being applied experimentally with success.[35] Epoxy injection is being used as a temporary repair of delaminated areas[36] or prior to cathodic protection to achieve more permanent results. The judicious application of waterproof coatings on a concrete containing corroding metal may serve to prolong the useful life of a structure, but their indiscriminate use may raise the level of moisture within the concrete and thus accelerate corrosion. None of the coatings offer permanent protection. Research on the removal of the chlorides which cause corrosion, and on deep impregnation of the concrete with polymer after drying, is underway and promising but not yet operational. Remedial measures of these types are recommended for use only after careful study by those versed in corrosion problems.

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CHAPTER 5—CHEMICAL REACTIONS OF AGGREGATES

5.1—Types of reactions

Chemical reactions of aggregates in concrete can affect the performance of concrete. Some reactions may be beneficial, but others result in serious damage to the concrete by causing abnormal expansion, cracking, and loss of strength.[1]

The reaction that has received greatest attention and which was the first to be recognized involves a reaction between alkalies (Na_2O and K_2O) from the cement, or from other sources, with hydroxyl, and certain siliceous constituents that may be present in the aggregate. This phenomenon was originally, and is still sometimes, referred to as "alkali-aggregate reaction," but in recent years it has been more properly designated as "alkali-silica reaction." The earliest paper discussing alkali-silica reaction, by Stanton,[2] appeared in 1940.

Deterioration of concrete has occurred in Kansas, Nebraska, and eastern Wyoming involving certain sand-gravel aggregates. The deterioration has been regarded as a chemical phenomenon.[3,4] Because early studies showed no consistent relationship between the distress and alkali content of the cement, as is normally the case with alkali-aggregate reactions, the term "cement-aggregate reaction" is used because it has been deemed desirable to distinguish between this phenomenon and the more widely occurring alkali-silica reaction. Subsequent research indicated that this phenomenon is a reaction between the alkalies in cement and some siliceous constituents of the aggregates, complicated by environmental conditions that produce high concrete shrinkage and concentration by drying.[5]

It has also been clearly demonstrated that certain carbonate rocks participate in reactions with alkalies which, in some instances, produce detrimental expansion and cracking. Detrimental reactions are usually associated with argillaceous dolomitic limestones which have somewhat unusual textural characteristics.[6] This reaction is designated as "expansive alkali-carbonate reaction." It has been extensively studied in Canada where it was originally recognized[7-12] and in the United States.[13-18]

In addition to the detrimental expansive alkali-carbonate reaction, a phenomenon associated with some carbonate rocks occurs wherein the peripheral zones of the aggregate particles in contact with cement paste are modified and develop prominent rims within the particle and extensive carbonation of the surrounding paste.[6,14,19-22] Some rims when etched with dilute acid appear in positive relief, while others exhibit negative relief; hence the terms "positive rims" and "negative rims" are commonly used. As contrasted with alkali-carbonate reactions which cause detrimental expansion and cracking, it is doubtful that rim-forming alkali-carbonate reaction is, per se, a deleterious reaction.[23]

In addition to the above types, other damaging chemical reactions involving aggregates should be mentioned. These include the oxidation or hydration of certain unstable mineral oxides, sulfates, or sulfides that occur after the aggregate is incorporated in the concrete (for example, the hydration of anhydrous magnesium oxide, calcium oxide, or calcium sulfate, or the oxidation of pyrite).[24] Metallic iron may occur as a contaminant in aggregate and subsequently be oxidized. Still other reactions may result from organic impurities (humus, sugar, etc.).[25] Engineers should be aware of these possibilities and supply corrective measures where necessary. Careful testing and the presence of such reactive impurities and their use in concrete can be avoided.

The alkali-silica, cement-aggregate, and expansive carbonate reactions are more important than the others and will be discussed in detail in the following section.

5.2—Alkali-silica reaction

5.2.1 *Occurrence*—A map[26] and data[27-37] are available showing areas known to have natural aggregates suspected of or known to be capable of alkali-silica reaction. Most of these references refer to North America; however, the available evidence[37] suggests that similar considerations are applicable elsewhere. Data are available for New Zealand, India, Germany, Iceland, Denmark, Turkey, and other countries.[37]

At the time of the 1960 report by Committee 201, it appeared that the greatest abundance of alkali-silica reactive rocks in the United States was in the western half of the country. This is probably still a correct estimate, if the kind of alkali-silica reaction intended is the quickly developing type which was the first to be recognized.[2,27,28,30] However, there is also a slowly developing type.[29]

The minerals, mineraloids, and rocks recognized as reactive in 1958 are shown in Table 5.2.1.[30] Since 1958, other rocks have been recognized as reactive. These include argillites, graywackes,[31] phyllites,[29] quartzites,[35] schists,[36] as well as fractured and strained quartz, recognized as reactive by L. S. Brown,[33] and granite gneiss.[34] Several of these rocks—including granite gneisses, metamorphosed subgraywackes, and some quartz and quartzite gravels—appear to react slowly even with high alkali cement, the reactivity not having been recognized until the structures were over 20 years old.[32-34]

Lightweight aggregates, being composed of predominantly amorphous silicates, would appear to have the potential for being reactive with cement alkalies. However, there is no evidence of distress of lightweight concrete caused by alkali reaction.[38]

5.2.2 *Mechanism*—Alkali-silica reaction can cause expansion and severe cracking of concrete structures and pavements. The phenomenon is complex and various theories have been advanced to explain field and laboratory evidence.[34,36,39-42] Unanswered questions remain. Apparently reactive material in the presence of potassium, sodium, and calcium hydroxide derived from the cement reacts to form either a solid nonexpansive calcium-alkali-silica complex, or an alkali-silica complex (also solid) which can expand by imbibition of

TABLE 5.2.1—DELETERIOUSLY REACTIVE ROCKS, MINERALS, AND SYNTHETIC SUBSTANCES

Reactive substance	Chemical composition	Physical character
Opal	$SiO_2 \cdot nH_2O$	Amorphous
Chalcedony	SiO_2	Microcrystalline to cryptocrystalline; commonly fibrous
Certain forms of quartz	SiO_2	(a) Microcrystalline to cryptocrystalline; (b) Crystalline, but intensely fractured, strained, and/or inclusion-filled
Cristobalite	SiO_2	Crystalline
Tridymite	SiO_2	Crystalline
Rhyolitic, dacitic, latitic, or andesitic glass or cryptocrystalline devitrification products	Siliceous, with lesser proportions of Al_2O_3, Fe_2O_3, alkaline earths, and alkalies	Glass or cryptocrystalline material as the matrix of volcanic rocks or fragments in tuffs
Synthetic siliceous glasses	Siliceous, with lesser proportions of alkalies, alumina, and/or other substances	Glass

The most important deleteriously alkali-reactive rocks (that is, rocks containing excessive amounts of one or more of the substances listed above) are as follows:

Opaline cherts	Andesites and tuffs
Chalcedonic cherts	Siliceous shales
Quartzose cherts	Phyllites
Siliceous limestones	Opaline concretions
Siliceous dolomites	Fractured, strained, and inclusion-filled quartz and quartzites
Rhyolites and tuffs	
Dacites and tuffs	

Note: A rock may be classified as, for example, a "siliceous limestone" and be innocuous if its siliceous constituents are other than those indicated above.

concentrations of alkali and calcium hydroxide, and on the available surface of the reactive material. When the alkali concentration is low enough, the initial product of reaction is nonexpansive; when the alkali concentration is high, the initial product of reaction is the expansive one. In the former case, for reaction to continue safely, the amount of reactive material must either be negligible or more than a certain amount, depending on the amount of alkali and fineness of reactive material. In the latter case, rapid reversal to a safe reaction (that is, formation of the nonexpansive product) is desirable and will occur if the reactive particles present sufficient surface for reaction, that is, if the reactive particles are sufficiently numerous or sufficiently fine.

5.2.3 *Laboratory tests for alkali-silica reactivity*—Laboratory tests should be made on aggregates from new sources and when service records indicate that reactivity may be possible. The most useful are:

(a) *Petrographic examination*—ASTM C 295 provides a recommended practice for the petrographic examination of aggregates. The types of minerals involved in alkali-aggregate reaction have been listed in Section 6.2.1, and procedures for recognizing these constituents have been described.[29,33,41,42] Recommendations are available which show the amounts of reactive minerals, as determined petrographically, which can be tolerated.[43-45] The procedures referenced above apply to reactive constituents recognized prior to 1960.

The reactive rocks and minerals that have been more frequently encountered since 1960 appear to have larger pessimum proportions and are harder to recognize in petrographic examination. Highly deformed quartz with an angle of undulatory extinction of 35 to 50 deg or more, and with deformation lamellae appear characteristic of the reactive quartz-bearing rocks. Relatively coarse-grained micas[35] have also been regarded as reactive constituents; fine-grained micas are reactive in argillities.[31]

(b) *Mortar bar test for potential reactivity (ASTM C 227)*—This method is the one most generally relied on to indicate potential alkali reactivity. Acceptance criteria are given by ASTM C 33 for evaluating these test results. The procedure is useful not only for the evaluation of aggregates, but also for the evaluation of specific aggregate-cement combinations. However, criteria have not been developed for the metamorphic siliceous and silicate rocks. From the results of Swenson, Gillott, and Duncan,[35] it may be expected that these rocks will not reliably develop expansive reaction in storage at 100 F (38 C) but will require more elevated temperatures and longer periods in test, probably 1 to 3 years, to develop evidence of reactivity. This prolongation of testing time makes it particularly desirable to employ petrographic criteria that will allow identification of these rocks.

(c) *Chemical test for potential reactivity (ASTM 289)*—This method is used primarily for a quick evaluation of natural aggregates, the results being obtainable in a few days as compared with 3 to 6 months or more with the mortar bar test. Acceptance criteria for this test are given in ASTM C 33 and elsewhere. Care must be exercised in interpreting the results of this test. Highway Research Board *Special Report* No. 31[30] and Highway Research Board *Bulletin* No. 239[46] give more details concerning proper interpretation of the results. Some of the reactive rocks identified since 1960 fall into a region below the end of the curve (Fig. 2, ASTM C 289) so that the results cannot be interpreted.

This test method has given questionable results when evaluating lightweight aggregates, and it is therefore not recommended for this purpose.[47]

5.2.4 *General criteria for judging reactivity*—When available, the field performance record of a particular aggregate, if it has been used with cement of high-alkali content, is the best means for judging its reactivity.[44] If such records are not available, the most reliable criteria are petrographic examination with corroborating evidence from the mortar bar test,[45] and sometimes supplemented by tests on concrete although these have not been standardized. The chemical test results should also be used in conjunction with results of the petrographic examination and mortar bar test. It is preferable not to rely on the results of only one kind of test in any evaluation.[45]

5.2.5 *Recommended procedures to be used with alkali-reactive aggregates*—If aggregates are shown by service records or laboratory examination to be potentially reactive, they should not be used when the concrete is to be exposed to seawater or alkali environments if nonreactive aggregates are available.[30] When reactive aggregates must be used, this should be done only after thorough tests, and preferably after service records have established that with appropriate limits on the alkali content of the cement, or with the use of appropriate amounts of an effective pozzolan, or both, satisfactory service can be anticipated.[48] In cases where seawater or alkaline soil environments are not involved and there are no sound materials available economically, reactive materials may be used provided the following safeguards are employed:

(a) *Low-alkali cement*—Specify a "low alkali" cement (maximum of 0.6 percent equivalent Na_2O). (Low alkali cement will become less readily available except at a premium price with the need to use less energy in manufacturing cement and with environmental control of cement plant emissions.) Prohibit the use of seawater or alkali soil water as mixing water and avoid addition of sodium or potassium chloride.

(b) *Pozzolan*—Where low-alkali cements are not economically available, use a suitable pozzolanic material as prescribed by ASTM C 618. Pozzolans should be tested in accordance with ASTM C 441 to determine their effectiveness in preventing excessive expansion due to the alkali-aggregate reaction. The criterion of 75 percent reduction based on an arbitrary cement-to-pozzolan ratio merely provides a basis of comparison. Pepper and Mather[48] showed that many pozzolans would need to be used at higher proportions to cement to achieve 75 percent reduction in expansion of a glass mixture with a cement having a 1.0 percent Na_2O equivalent. Fortunately, most reactive aggregates are less reactive than glass. Whenever the use of pozzolanic materials is considered, it should be remembered that if these materials increase water demand, they may cause increased drying shrinkage in concrete exposed to drying. Increased water demand results from high fineness and poor particle shape. The rate of strength development in correctly proportioned pozzolanic concrete can equal that of portland cement concretes at 28 days.

5.3—Cement-aggregate reaction

5.3.1 *Occurrence*—Sand-gravel aggregates in the Kansas, Nebraska, and Wyoming areas, especially those from the Platte, Republican, and Laramie river areas, have been involved in concrete deterioration attributed to cement-aggregate reaction.[3-5]

5.3.2 *Mechanism*—Recent research indicates that the cement-aggregate reaction is mainly a reaction between the alkalies in the cement that produce high pH and abundant hydroxyl and siliceous constituents of the aggregates. However, the field performance of concretes made with reactive sand-gravels does not correlate well with cement alkali content. The concrete deterioration results from moderate interior expansion caused by alkali-silica reactivity, and surface shrinkage caused by severe drying conditions in areas such as western Kansas and Nebraska. Evaporation at the surface of the concrete causes an increase in alkali concentration in the pore fluids near the drying surface, and a net migration of alkali toward this surface. Under these conditions even a low-alkali cement may cause objectionable deterioration, particularly near the surface. This alkali distribution is altered by the leaching of alkalies near the surface during periods of heavy rain.[5]

5.3.3 *Identification by laboratory tests*—Although special tests, such as ASTM C 342, have been devised to indicate potential damage from this phenomenon, their reliability is doubtful. Petrographic examination (ASTM C 295) and mortar bars (ASTM C 227), with the results interpreted as described by Hadley,[5] are regarded as more reliable.

5.3.4 *Recommended procedure to be employed with potentially deleterious cement-aggregate combinations*—The use of potentially deleterious cement-aggregate combinations should be avoided where possible. However, if they must be used, a suitable pozzolan that does not increase drying shrinkage and 30 percent or more (by weight) of coarse limestone should be used with potentially deleterious cement-aggregate combinations. Concrete tests should be used to determine whether the resulting combination is satisfactory,[30,49] and whether the limestone is frost resistant in air-entrained concrete in the grading in which it is used.

5.4—Expansive alkali-carbonate reactivity

5.4.1 *Occurrence*—Certain limestone aggregates, usually dolomitic, have been reported as reactive in concrete structures in Canada (Ontario) and in the United States (Illinois, Indiana, Iowa, Michigan, Missouri, New York, South Dakota, Virginia, and Wisconsin). Both quarried aggregates and gravels from the same formation may be reactive.

5.4.2 *Mechanism*—Many unanswered questions remain, and more than one mechanism to explain expansive carbonate reactivity has been proposed.[6,10,11,13,16] It is clear that dedolomitization leading to the formation of brucite and the regeneration of alkali occurs. This is a distinguishing feature from alkali-silica reactivity in which the initial alkali is used up as the reaction proceeds. The presence of clay minerals appears significant and their swelling, when opened to moisture by dedolomitization, is the basis for one of the possible explanations of the reaction.[11]

Rim growth is not unusual in many carbonate rocks, and it has been reported as associated with distress in pavements in Iowa.[50] However, this is not always the case. The nature of rim formation is not fully understood.[6] It is, however, associated with a change in the disposition of silica and carbonate between the aggregate particle and the surrounding cement paste, the rims appearing to extend concentrically deeper into the aggregate with time.

The affected concrete is characterized by a network of pattern or map cracks usually most strongly developed in areas of the structure where the concrete has a constantly renewable supply of moisture, such as close to the waterline in piers, from the ground behind retaining walls, beneath road or sidewalk slabs, or by wick action in posts or columns. A distinguishing feature from alkali-silica reaction is the general absence of silica gel exudations at cracks. Additional signs of the severity of the reaction are closed expansion joints with possible crushing of the adjacent concrete.[6,54]

5.4.3 *Identification by laboratory tests*

(a) *Petrographic examination of aggregate*—Such examination may be used to identify the features of the rock, as listed by Hadley,[6] and modified by Buck[51] and Dolar-Mantuani.[52,53] While it is generally true that reactive rocks can be characterized as having dolomitic rhombs from 1 to 200 μm in maximum dimension in a background of finer calcite and insoluble residue, the presence of all or any dolomite in a fine-grained* carbonate rock makes it desirable to make the rock-cylinder test (ASTM C 586). This is recommended whether or not the texture is believed to be typical, and whether or not insoluble residue including clay amounts to a substantial portion of the aggregate. As expansive rocks are recognized from more areas, the more variable the textures and compositions appear to be.

(b) *Expansion of concrete prisms*—The prisms are made with job materials and stored at 100 percent relative humidity at 73 F (23 C),[54] or (in order to accelerate the reaction) they may be made with additional alkali and/or stored at elevated

*Fine-grained is generally regarded as 1 mm and finer. However, as a precautionary measure rocks with grains 2 mm and finer should be tested in rock cylinders.

temperature.[55-57] Comparison is usually made with the expansion of prisms containing a nonreactive control aggregate.

(c) *Dilation of the aggregate in finely powdered form in the presence of alkali in the Powder Cell Test*[57]

(d) *Petrographic analysis of the concrete*—This can confirm the type of aggregate present and its characteristics as outlined in Section (a) above. Distress that has occurred in the aggregate and surrounding matrix, such as micro- and macro-cracking, may be observed. Reaction rims may be observed in certain aggregate particles and may be identified as negative or positive by acid etching. They do not necessarily signify harmful results. Secondary deposits of calcium carbonate, calcium hydroxide, and ettringite may be found in voids within the concrete. Deposits of silica, hardened or in gel form, associated with the suspect aggregate pieces will not be found.[6]

(e) *Other laboratory tests*—Alkali-carbonate reaction may be identified by visual observation of sawed or ground surfaces. X-ray examination of reaction products is also sometimes useful.*

5.4.4 *Criteria for judging reactivity*—Definitive correlations between expansions occurring in the laboratory in rock cylinders or concrete prisms and deleterious field performance have not yet been established. The factors involved are complex and include the heterogeneity of the rock, coarse aggregate size, permeability of the concrete, and seasonal changes in environmental conditions in service, principally availability of moisture, level of temperature, and possibly the use of sodium chloride as a deicing chemical.

Cracking is usually observed in concrete prisms at an expansion of about 0.05 percent. Experience in Ontario[54,56] indicates that if concrete prisms with the proposed combination of job materials stored at 73 F (23 C) at 100 percent relative humidity do not show expansion greater than 0.02 percent before 84 days, harmful reactivity is unlikely. Slightly less restrictive criteria have been suggested elsewhere.[56]

It is not certain that rapid determination of potential reactivity can always be made by using the rock cylinder test, since some rocks showing an initial contraction may develop considerable expansion later on.[52,59] No universal correlation exists between the expansion of rock cylinders and concrete in service, though it may exist with concrete prisms stored in the laboratory.[6,14,58]

Expansions greater than 0.10 percent in the rock cylinders are usually taken as a warning that further tests should be undertaken to determine expansion of the aggregate in concrete. Fortunately many carbonate rocks that expand in rock cylinders do not expand in concrete.

5.4.5 *Recommended procedures to minimize alkali-carbonate reactivity*—Procedures that can be employed to mitigate the effects of the reaction include:

(a) Avoiding reactive rocks by selective quarrying[19,55,57]

(b) Dilution with nonreactive aggregates, or use of a smaller maximum size.[14,54]

(c) Use of low alkali cement (probably 0.4 percent combined alkali or lower). This will prevent harmful expansions in most cases;[54,58] however, in pavements where sodium chloride is used as a deicing chemical, this cannot be taken as certain.[55,58]

Of these measures, the first is the safest and usually the most economical.

5.5—Preservation of concrete containing reactive aggregate

There are no known methods of adequately preserving existing concrete which contains the elements that contribute to the previously described chemical reactions. Water or moisture is partly involved in at least two of these reactions. The destructive effects of freezing and thawing are more pronounced after the initial stages of destruction by these chemical reactions. Therefore, any practicable means of decreasing the exposure of such concrete to water may extend its useful life.

5.6—Recommendations for future studies

Current criteria employed in the United States that provide a basis for separating aggregates into "reactive" and "nonreactive," while generally effective in preventing recurrences of catastrophic destruction of concrete structures, are now seen to be inefficient in two ways. First, they have caused more severe precautions to have been taken (limiting calculated cement alkalies to 0.60 percent Na_2O equivalent when a higher maximum would surely have been "safe") than were justified. Second, they have sometimes permitted alkali-silica reaction to occur to a degree causing notable cracking when aggregates classed as "nonreactive" were used with cements containing more than 0.60 percent Na_2O equivalent.

It is concluded that new research, or a reinterpretation of the results of previous research, is needed to better characterize the following relevant parameters:

*ASTM C 227, C 289, and C 342 (applicable to alkali-silica reaction) are not applicable to expansive carbonate reactivity

(a) Degree and rate of aggregate reactivity

(b) Influence of concrete mixture proportions, especially unit cement content

(c) Influence of environment on the concrete

(d) Influence of dimensions of structures

If these parameters were better understood, one could develop the sort of prescription for safe structural behavior that would serve efficiently to prevent damage to concrete from reactions between aggregates and alkalies. This prescription might be a sort of nomograph where one selected a point on a scale of low to high aggregate reactivity, a point on a scale of cement content, a point on a scale of structural dimensions, a point on a scale of environmental exposure (temperature, moisture), and by connecting these one could be directed to a point on a scale of degree of precaution to take. Then one could work from the other side of the coin, taking the type and amount of slag or pozzolan in the cementitious medium, the $Na_2O:K_2O$ ratio in the cement, the ratio of water-soluble to total alkali, and finally establish the limit on alkali in the cement appropriate for the concrete to be used in a given structure, in a given location, to be constructed with aggregate from a given source.

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CHAPTER 6—REPAIR OF CONCRETE

Detailed coverage of concrete repairs falls within the mission of ACI Committee 546.[1] A guide for the repair of reinforced concrete bridges is now in preparation, and other reports will follow. This chapter will therefore give only a brief, general coverage of the subject, with emphasis on the durability aspect.

6.1—Evaluation of damage and selection of repair method

To objectively evaluate the damage to a structure, it is necessary to determine what caused the damage in the first place. The damage may be the result of poor design, faulty workmanship, mechanical abrasive action, cavitation or erosion from hydraulic action, leaching, chemical attack, chemical reaction inherent in the concrete mixture, exposure to deicing agents, corrosion of embedded metal, or other lengthy exposure to an unfavorable environment. Guidance for examining and sampling concrete in construction may be found in ASTM C 823.

Whatever may have been the cause, it is essential to establish the extent of the damage, and determine if the major portion of the structure is of suitable quality on which to build a sound repair. Based on this information, the type and extent of the repair are chosen. This is the most difficult step—one which requires a thorough knowledge of the subject and mature judgment by the engineer. If the damage is the result of moderate exposure of what was an inferior concrete in the first place, then replacement by good quality concrete should assure lasting results. On the other hand, if good quality concrete was destroyed, the problem becomes more complex. In that case, a very superior quality of concrete is required, or the exposure conditions must be altered.

The repair of spalls from reinforcing bar corrosion (see Section 4.6) requires a more detailed study. Simply replacing the deteriorated concrete and restoring the original cover over the steel will not solve the problem. Also, if the structure is salt-contaminated, the electrolytic conditions will be changed by the application of new concrete, and the consequences of these changed conditions must be considered before any repairs are undertaken.

6.2—Types of repairs

6.2.1 *Concrete replacement*—The concrete replacement method consists of replacing defective concrete with machine-mixed concrete of suitable proportions and consistency, so that it will become integral with the base concrete.

Concrete replacement is the desired method if there is honeycomb in new construction or deterioration of old concrete which goes entirely through the wall or beyond the reinforcement, or if the quantity is large. For new work, the repairs should be made immediately after stripping the forms.[2,3] Considerable concrete removal is always required for this type of repair. Excavation of affected areas should continue until there is no question that sound concrete has been reached. Additional chipping may be necessary to accommodate the repair method and shape the cavity properly.

Concrete for the repair should generally be similar to the old concrete in maximum size of aggregate and water-cement ratio.

Forming will usually be required for large repairs in vertical surfaces.

6.2.2 *Dry pack*—The dry pack method consists of ramming a very stiff mix into place in thin layers. It is suitable for filling form tie-rod holes and narrow slots, and for repairing any cavity which has a relatively high ratio of depth to area. Practically no shrinkage will occur with this mix, and it develops a strength equalling or exceeding that of the parent concrete. The method does not require any special equipment, but cement finishers must be trained in this type of repair if the results are to be satisfactory.[3]

6.2.3 *Preplaced aggregate concrete*—Preplaced aggregate concrete may be used advantageously for certain types of repairs. It bonds well to concrete and has low drying shrinkage. It is also well adapted to underwater repairs. This is a specialized process which is described elsewhere.[4]

6.2.4 *Shotcrete*—Shotcrete has excellent bond with new or old concrete and is frequently the most satisfactory and economical method of making shallow repairs. It is particularly adapted to vertical or overhead surfaces where it is capable of supporting itself (without a form) without sagging or sloughing. Shotcrete repairs generally perform satisfactorily where recommended procedures are followed.[5] Simplified equipment has been developed for use in small repairs.[3]

6.2.5 *Repair of scaled areas and spalls in slabs*—Scaling of concrete pavement surfaces is not unusual where they are subject to deicing salts, particularly if the concrete is inadequately air-entrained. Such areas may be satisfactorily repaired by a thin concrete overlay provided the surface of the old concrete is sound, durable, and clean.[6,7] A minimum overlay thickness of about 1½ in. (38 mm) is needed for good performance. The temperature of the underlying slab should be as close as possible to that of the new concrete.

Spalls may occur adjacent to pavement joints or cracks. Spalls usually are several inches in depth, and even deeper excavation may be required to remove all concrete which has undergone some slight degree of deterioration. They may be repaired by methods similar to those used for scaled areas.

Numerous quick setting patching materials, some of which are proprietary, are available. Information on the field performance of these materials is given in Reference 8.

6.3—Preparations for repair

Sawcuts around the perimeter of a repair are usually advisable, particularly in the case of slabs, to eliminate feather edges. If practicable, the sawcuts should be made at a slight angle so that the width at the base of the patch is greater than at the surface, thereby providing some keying action.

All deteriorated or defective concrete must be removed; in the case of slabs, suitable mechanical scarification equipment should be used. Next, the surfaces of the concrete must be thoroughly cleaned, preferably by wet sandblasting.

The bonding surface should have been previously wet down, but should be dry at the time of patching. The dry surface should be carefully coated with a layer of mortar about ⅛ in. (3 mm) thick, or with another suitable bonding agent (see Section 6.4). The repair should proceed immediately.

6.4—Bonding agents

Bonding agents are used to establish unity between fresh concrete or mortar and the parent concrete. Sand-cement mortar or neat cement paste has generally been used in the past. Many reports in the literature testify to the success of these treatments where recommended practices have been followed.

Epoxy resin is now used considerably as a bonding agent, with the expectation of durable results.[9] This material develops a bond having greater tensile, compressive, and shear strength than concrete. It is waterproof and highly resistant to chemical and solvent action. It is possible to have acceptable results when the concrete is brought to a feather edge; better results, however, are obtained if a 1-in. (25 mm) minimum thickness is maintained. There are some disadvantages in using epoxy resin, including its high cost, toxicity, and short pot life.

Other types of bonding agents have recently become available. Certain latexes, supplied as an emulsion or dispersion, improve the bond and have good crack resistance. Polyvinyl acetates, stryrene-butadiene, and acrylic are among those used. These materials, particularly the polyvinyl acetates, must be properly compounded if the dried film is to be resistant to moisture. They may be used either as a bonding layer or added to the concrete or mortar mix.

6.5—Appearance

Unless proper attention is given to all of the factors influencing the appearance of concrete repairs, they are likely to be unsightly. In concrete where appearance is important, particular care should be taken to insure that the texture and color of the repair will match the surrounding concrete. A proper blend of white cement with the job cement will enable the patch to come close to matching the color of the original concrete. A patch on a formed concrete surface should never be finished with a steel trowel, since this produces a dark color which is impossible to remove.

6.6—Curing

All patches (except where epoxy mortar or epoxy concrete is used) must be properly cured to assure proper hydration of the cement and durable concrete or mortar. The recommendations of ACI Committee 308 should be followed.[10]

6.7—Treatment of cracks

The decision of whether a crack should be repaired to restore structural integrity or merely sealed is dependent on the nature of the structure and the cause of the crack, and upon its location and extent. If the stresses which caused the crack have been relieved by its occurrence, the structural integrity can be restored with some expectation of permanency. However, in the case of working cracks (such as cracks caused by foundation movements, or cracks which open and close from temperature changes), the only satisfactory solution is to seal them with a flexible or extensible material.

Thorough cleaning of the crack is essential before any treatment takes place. All loose concrete, oil joint sealant, and other foreign material must be removed. The method of cleaning is dependent upon the size of the crack and the nature of the contaminants. It may include any combination of the following: compressed air, wire brushing, sandblasting, routing, or the use of picks or similar tools.

Restoration of structural integrity across a crack has been successfully accomplished using pressure and vacuum injection of low viscosity epoxies[11,12] and other monomers[13] which polymerize in situ and rebond the parent concrete.

Sealing of cracks without restoration of structural integrity requires the use of materials and techniques similar to those used in sealing joints. A detailed discussion of the types of joint sealant available and methods of installation is contained in ACI Committee 504 report, "Guide to Joint Sealants for Concrete Structures."[14] Since cracks are generally narrower than joints, some modification in procedure, such as widening the crack with a mechanical router or the use of a low viscosity material, is often necessary.

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CHAPTER 7—USE OF COATINGS TO ENHANCE CONCRETE DURABILITY

Coatings are being used increasingly for the purpose of enhancing concrete durability. Their proper use requires an understanding of their advantages and limitations.[1] It is the intent of this chapter to outline only the general principles involved.

Table 7 lists several classes of coatings which may be considered for different purposes and environments. Class I (surface water repellents), and II and III (plastic and elastomeric coatings), are discussed below.

7.1—Surface water repellents

Surface water repellents (Class I in Table 7) may be used on concrete pavements to prevent or minimize scaling from the use of deicers (see Section 1.4.6). This is a low cost treatment that many have found provides a degree of protection for non-air-entrained concrete, or is added insurance for air-entrained concrete placed in the fall which is subjected to deicing salts during the first winter. It may also be used to prolong the service life of

TABLE 7—COATINGS FOR CONCRETE IN VARIOUS ENVIRONMENTS

Class No.	Type and thickness of coating	Materials	Description of environment or purpose of coating
I	Surface water repellent—under 5 mils (0.13 mm)	Linseed oil, polyvinyl butyral, silicone	Water repellency. Superficial sealing of surface. Protection against deicing salts.
II	Plastic and elastomeric—5 to 50 mils (0.13 to 1.3 mm)	Epoxy, polyurethane, asphalt, coal tar, chlorinated rubber	Improve freeze-thaw resistance. Seal surface for high-purity water service. Protect concrete in contact with solutions having a pH as low as 3.
III	Plastic and elastomeric—50 to 250 mils (1.3 to 6.4 mm)	Glass reinforced epoxy, glass reinforced polyester, sheet neoprene, spray applied neoprene	Protect concrete tanks during continuous exposure to dilute mineral and organic acids up to 158 F (70 C)
		Sand filled epoxy, sand filled polyester	Protect concrete floors during intermittent exposure to dilute acids. Also to protect floors in dairy and food processing industries.
IV	Composite systems—over 250 mils (6.4 mm)	Asphalt membrane covered with acid proof tile, or brick laid with a chemical resistant mortar*	Protect concrete floors from concentrated acids or acid/solvent combinations. Good for liquids up to 212 F (100 C)

*See Reference 7.

older pavements. A number of materials have been tested, some being more effective than others.[2-4] Most studies indicate that linseed oil is the best choice when both effectiveness and cost are considered.

A mixture of 50 percent boiled linseed oil and 50 percent mineral spirits is normally used. It should be put on in two applications when the concrete surface is dry and clean. For estimating purposes, a coverage of 40 sq yd (33 sq m) per gallon for the first application and 65 sq yd (54 sq m) per gallon for the second application may be assumed. However, experience has shown that because of varying porosities of different concretes, the actual application rate should be determined from a test strip on each pavement. Applications which are too light or too heavy are to be avoided; both are ineffective in preventing scaling and the latter also adversely affects skid resistance. A linseed oil treatment should provide temporary protection (for 1 to 3 years), after which another application may be made if needed.

Another surface water repellent, silicone, has sometimes been used on concrete or masonry walls—mainly to minimize moisture penetration which in turn can affect durability. Results have not always been good, especially where moisture has access to the back side of the wall and carries dissolved salts to the front face. Also, silicone oxidizes rapidly due to ozone in the atmosphere, and is somewhat water soluble. Retreatment is required every 1-5 years.

7.2—Plastic and elastomeric coatings

7.2.1 *Materials*—Plastic and elastomeric coatings are capable of forming a strong, continuous film. Thirty-nine generic types are listed in a report by ACI Committee 515.[5] The most promising ones for application to concrete are listed in Classes II and III in Table 7. The selection of the coating is based largely on the type of environment, and on its severity or aggressiveness. Those in general used to protect concrete against chemical attack are included in the table, as well as others sometimes used to protect against abrasion or to minimize damage from freeze-thaw cycles. To be effective in protecting concrete, the coatings must have certain basic properties:

1. The adhesive bond strength of the coating (to the concrete) must be at least equal to the tensile strength of the concrete at the surface.

2. The abrasion resistance must be adequate to prevent the coating from being removed.

3. Where they are in a chemical environment, the chemicals must not cause swelling, dissolving, cracking, or embrittlement of the material. Nor should the chemicals permeate or diffuse through the coating so as to destroy the adhesion between the coating and concrete.

There is no guarantee that coatings made by different manufacturers will perform the same, even where classified as the same generic type. Coatings vary in the types and amounts of ingredients, so their performance will also vary. In addition, the application characteristics, particularly the ease of applying a coating to concrete, will affect its performance.

The type and thickness of coating required will depend on the severity or aggressiveness of the environment. Coating selection must be based on testing or past experience. Because there are no standard test methods, the most reliable procedure is to subject the entire coating system to the environmental conditions that will be encountered in service as closely as possible. If the selection of the coating must be made before tests of sufficient duration (6 to 12 months minimum) can be conducted, the coating supplier should be asked to supply fully documented case histories where his coating system has protected concrete under the same or similar environmental conditions. The selection of a reliable coating supplier is as important as the selection of the coating itself.

7.2.2 *The coating as part of a system*—To understand the behavior of a plastic or elastomeric coating, it is necessary to consider the coating not as an isolated material, but as part of a system. The elements of a coating system for concrete are shown in Fig. 7.2.2., and the role of each is explained below. Although this analysis is aimed particularly at slabs-on-grade, the basic principles will apply to many other concrete structures.

7.2.2.1 *Concrete-coating interface.* Most coating materials specifically formulated for use over concrete develop and maintain an adhesive bond strength greater than the tensile strength of the concrete. For this reason, adhesion is not a major problem affecting the performance of a coating provided it is applied to a clean, dry surface. The surface must be free of (1) loose particles of dirt and dust, (2) oil or other chemicals that prevent adhesion, and (3) water. If the surface is not clean, alkaline washing, acid etching, or sandblasting may be employed. If these methods prove unsatisfactory, all of the contaminated material at or near the surface should be removed by a scarifier or jackhammer. Certain coatings have the ability to bond to damp surfaces, but their long-term adhesion is questionable.

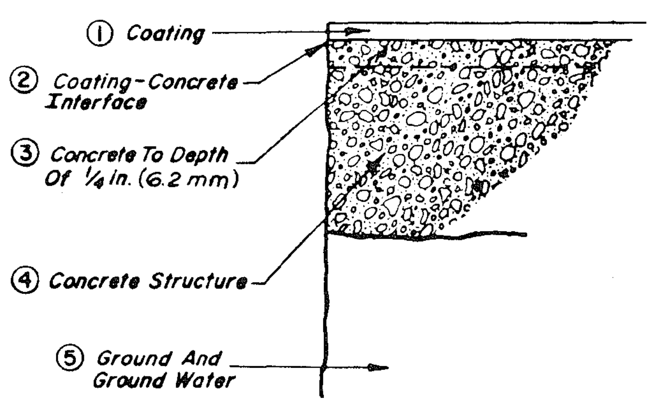

Fig. 7.2.2—Elements of a coating system for concrete

Not only is surface moisture objectionable, but moisture within the concrete may affect the ability of a coating to adhere to the surface.[6] There are no precise guidelines to indicate when moisture will be a problem, although a qualitative test is available. A brief explanation of how the moisture in concrete can affect the adhesion of a coating is as follows:

Poor adhesion between concrete and a coating can result if water vapor diffusing out of the concrete condenses at the concrete/coating interface before the coating has had an opportunity to cure. Whether this will be a problem depends on (1) the rate of vapor transmission through the concrete, and (2) the temperature gradient between concrete and air while the coating is curing; if the concrete temperature is below that of the air, there is less likelihood that water vapor will condense.

The qualitative moisture test (for normal weight concrete) is conducted as follows:

1. Tape a polyethylene film [0.006 in. (0.15 mm) x 4 ft x 4 ft (1.25 m x 1.25 m)] over the concrete surface.

2. After the film has been in place for 12 hr, determine whether moisture has condensed on the underside. (This is the time required for many epoxy coatings to develop a partial cure of 30 percent.)

3. If condensation is noted, additional drying of the concrete is required. Another moisture test should then be made to insure that the moisture content has dropped to an acceptable level before applying the coating.

Another problem is air bubbles of varying sizes in the concrete surface, which may prevent the coating from forming a continuous and impervious film. Prior to its application, a heavily bodied, thixotropic plastic mortar should be spread over the surface to fill or bridge over these defects. Coating suppliers can furnish such materials, which should be compatible with the coating.

7.2.2.2 *Concrete to a depth of ¼ in. (6.4 mm).* This is the most critical part of the coating system. When a coating fails, a thin layer of concrete, usually less than ⅛ in. (3.2 mm) thick, generally adheres to the underside of the coating. This means that the concrete failed because the internal stresses in the coating were greater than then tensile strength of the concrete near the interface. These stresses are derived from two sources:

1. Shrinkage and locked-up stresses when the material was cured. This is common to all two-component coatings cured by a chemical reaction between the resin and curing agent.

2. Differential volume change of the concrete and the coating because of a difference in temperature and, more importantly, of a difference in coefficient of thermal expansion. Most coatings have a much higher temperature coefficient than concrete.

Weakness of the concrete near the surafce can be caused by overworking during finishing, the presence of laitance on the surface, or by improper curing. On the other hand, high strength coatings applied in thick layers may cause even sound concrete to fail. Low modulus coatings develop lower stresses and are recommended where it is anticipated that fairly large stresses will develop in service.

7.2.2.3 *Concrete structure.* The concrete section can destroy the ability of the coating to protect it. Any cracks in the concrete which occur or enlarge after the coating has been applied will reflect through the coating. A poor quality concrete slab with high permeability may allow ground water to travel through the concrete so rapidly that the surface will never dry sufficiently to accept a coating, or it may push the coating away from the concrete later.

7.2.2.4 *Foundation conditions.* A dimensionally unstable base or one that does not have sufficient supporting strength can cause cracks in the concrete which are detrimental to coatings, as discussed above. Also, the availability and amount of ground water is a major factor in the success of a coating. The use of an impermeable membrane to prevent the entry of water into the concrete is advisable where possible.

7.2.3 *Precautions when using coatings to minimize freeze-thaw damage*—The intent of coatings here is to keep the moisture content of the concrete below the critical saturation point so that it will not sustain damage during freezing. Such coatings should be used with extreme caution, particularly on slabs or walls which are exposed continuously to moisture on the back side. The indiscriminate use of impervious coatings under these conditions may actually trap water in the concrete and adversely affect durability. One might expect better results from coating a bridge pier cap (perhaps to protect the concrete against deicing salts) provided the concrete was dry at the time the coating was applied. Breathable coatings, which are claimed to keep additional water from penetrating a concrete section, while allowing vapor to escape, are available and show some promise.

7.3—Future of coatings

It is to be expected that coatings will play an increasingly important role in protecting concrete in the future, as more knowledge on their properties and performance becomes available. Considerable research is underway in this area, including many experimental field applications.

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   ASTM C 386-71, Use of Chemical-Resistant Sulfur Mortars

   ASTM C 397-67, Use of Chemically Setting Chemical-Resistant Silicate and Silica Mortars

   ASTM C 399-67, Use of Chemical-Resistant Rosin Mortars

This report was submitted to letter ballot of the committee, which consists of 24 members, all of whom returned their ballots and voted affirmatively.

For discussion see ACI JOURNAL, *Proceedings* V. 75, No. 6, June 1978, pp. 269-273.

ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction, and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be incorporated directly into the Project Documents.

ACI 201.1R-68

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Guide for Making a Condition Survey of Concrete in Service

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This guide provides a system for reporting on the condition of concrete in service. It includes a check list of the many details to be considered in making a report, and provides standard definitions of 40 terms associated with the durability of concrete. Its purpose is to establish a uniform system for evaluating the condition of concrete.

Keywords: buildings; concrete construction; concrete durability; concrete pavements; concretes; corrosion; cracking (fracturing); deterioration; environment; freeze-thaw durability; inspection; joints; popouts; quality control; scaling; serviceability; spalling; strength; surveys (data collection).

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■ A CHECK LIST IS provided for making a survey of the condition of concrete. The list is designed to be used in recording the history of a concrete project from inception through completion and subsequent life of the structure or pavement.

While it probably will be used most often in connection with the survey of concrete that is showing some degree of distress, its application is recommended for all important concrete structures. In any case, records of the materials and construction practices used should be maintained because they are difficult to obtain at a later date.

The committee has attempted to include all pertinent items that might have a bearing on the performance of the concrete. However, those making the survey should not limit their investigation to the items listed, thereby overlooking or ignoring other possible contributing factors. Simply following the guide will not eliminate the need for intelligent observation and the use of sound judgment.

Those performing the survey should be experienced and competent in this field. In addition to verbal descriptions, numerical data obtained by laboratory tests and field measurements should be provided wherever possible. Photographs, including a scale to indicate linear dimensions, are of great value in showing condition of structure.

One of the objects of a condition survey is to provide information that will be of value in the construction of more economical, serviceable structures. The survey may show causes of deterioration or lack of need of expensive materials or construction methods. The check list should be used in conjunction with the following:

1. ACI Committee 311, "Recommended Practice for Concrete Inspection (ACI 311-75)," American Concrete Institute, 1975, 6 pp. Also *ACI Manual of Concrete Practice*, Part 2.
2. ACI Committee 201, "Guide to Durable Concrete"—ACI 201.2R-77, ACI JOURNAL, *Proceedings* V. 74, No. 12, Dec. 1977, pp. 573-609. Also *ACI Manual of Concrete Practice*, Part 1.

CHECK LIST

1. Description of structure or pavement

1.1. Name, location, type, and size

1.2. Owner, project engineer, contractor, when built

1.3. Design

1.3.1. Architect and/or engineer

1.3.2. Intended use and history of use

1.3.3. Special features

1.4. Photographs

1.4.1. General view

1.4.2. Detailed close-ups of condition of area

1.5. Sketch map — orientation showing sunny and shady walls and well and poorly drained regions

2. Present condition of structure

2.1. Over-all alignment of structure

2.1.1. Settlement

2.1.2. Deflection

2.1.3. Expansion

2.1.4. Contraction

2.2. Portions showing distress (beams, columns, pavement, walls, etc. Subjected to strains and pressures)

2.3. Surface condition of concrete

2.3.1. General (good, satisfactory, poor, etc.)

2.3.2. Cracks

2.3.2.1. Location and frequency

2.3.2.2. Type and size

2.3.2.3. Leaching, stalactites

2.3.3. Scaling

2.3.3.1. Area, depth

2.3.3.2. Type (see definition)

2.3.4. Spalls and popouts

2.3.4.1. Number, size and depth

2.3.4.2. Type (see definitions)

2.3.5. Extent of corrosion or chemical attack

2.3.6. Stains

2.3.7. Exposed steel

2.3.8. Previous patching or other repair

2.4. Interior condition of concrete

2.4.1. Strength of cores

2.4.2. Density of cores

2.4.3. Moisture content (degree of saturation)

2.4.4. Evidence of alkali-aggregate or other reaction

2.4.5. Bond to aggregate, reinforcing steel, joints

2.4.6. Pulse velocity

2.4.7. Volume change

2.4.8. Air content and distribution

3. Nature of loading and detrimental elements

3.1. Exposure

3.1.1. Environment — arid, subtropical, marine, freshwater, industrial, etc.

3.1.2. Weather — (July and January mean temperatures, mean annual rainfall and months in which 60 percent of it occurs)

3.1.3. Freezing and thawing

3.1.4. Wetting and drying

3.1.5. Drying under dry atmosphere

3.1.6. Chemical attack — sulfates, acids

3.1.7. Abrasion, erosion, cavitation

3.1.8. Electric currents

3.2. Drainage

3.2.1. Flashing

3.2.2. Weepholes

3.2.3. Contour

3.3. Loading

3.3.1. Dead

3.3.2. Live

3.3.3. Impact

3.3.4. Vibration
3.3.5. Traffic index
3.3.6. Other
3.4. Soils (foundation conditions)
3.4.1. Stability
3.4.2. Expansive soil
3.4.3. Settlement
3.4.4. Restraint

4. Original condition of structure
4.1. Condition of formed and finished surfaces
4.1.1. Smoothness
4.1.2. Air pockets
4.1.3. Sand streaks
4.1.4. Honeycomb
4.1.5. Soft areas
4.2. Early structural defects
4.2.1. Cracking
4.2.1.1. Plastic shrinkage
4.2.1.2. Settlement
4.2.1.3. Cooling
4.2.2. Curling
4.2.3. Structural settlement

5. Materials of construction
5.1. Hydraulic cement
5.1.1. Type and source
5.1.2. Chemical analysis (obtain certified test data if available)
5.1.3. Physical properties
5.2. Aggregates
5.2.1. Coarse
5.2.1.1. Type, source and mineral composition (representative sample available)
5.2.1.2. Quality characteristics
5.2.1.2.1. Percentage of deleterious material
5.2.1.2.2. Percentage of potentially reactive materials
5.2.1.2.3. Coatings, texture, and particle shape
5.2.1.2.4. Gradation, soundness, hardness
5.2.1.2.5. Other properties as specified in ASTM Designation C 33 (C 330 — for lightweight aggregate)
5.2.2. Fine aggregate
5.2.2.1. Type, source, and mineral composition (representative sample available)
5.2.2.2. Quality characteristics
5.2.2.2.1. Percentage of deleterious material
5.2.2.2.2. Percentage of potentially reactive materials
5.2.2.2.3. Coatings, texture and particle shape
5.2.2.2.4. Gradation, soundness and hardness
5.2.2.2.5. Other properties as specified in ASTM Designation C33 (C330 for lightweight aggregate)
5.3. Mixing water
5.3.1. Source and quality
5.4. Air-entraining agents
5.4.1. Type and source
5.4.2. Composition
5.4.3. Amount
5.4.4. Manner of introduction
5.5. Admixtures
5.5.1. Mineral admixture
5.5.1.1. Type and source
5.5.1.2. Physical properties
5.5.1.3. Chemical properties
5.5.2. Chemical admixture
5.5.2.1. Type and source
5.5.2.2. Composition
5.5.2.3. Amount
5.6 Concrete
5.6.1. Mixture proportions
5.6.1.1. Cement content
5.6.1.2. Proportions of each size aggregate
5.6.1.3. Water-cement ratio
5.6.1.4. Water content
5.6.1.5. Chemical admixture
5.6.1.6. Mineral admixture
5.6.1.7. Air-entraining agent
5.6.2. Properties of fresh concrete
5.6.2.1. Slump
5.6.2.2. Percent air
5.6.2.3. Workability
5.6.2.4. Unit weights
5.6.2.5. Temperature
5.6.3. Type
5.6.3.1. Cast-in-place
5.6.3.2. Precast
5.6.3.3. Prestressed
5.6.4. Reinforcement
5.6.4.1. Yield strength
5.6.4.2. Thickness of cover
5.6.4.3. Presence of stirrups
5.6.4.4. Use of welding

6. Construction practices
6.1. Storage and processing of materials
6.1.1. Aggregates
6.1.1.1. Grading
6.1.1.2. Washing
6.1.1.3. Storage
6.1.1.3.1. Stockpiling
6.1.1.3.2. Bins
6.1.2. Cement and admixtures
6.1.2.1. Storage
6.1.2.2. Handling
6.1.3. Reinforcing steel and inserts
6.1.3.1. Storage
6.1.3.2. Placement
6.2. Forming
6.2.1. Type
6.2.2. Bracing
6.2.3. Coating
6.2.4. Insulation

6.3. Concreting operation
6.3.1. Batching plant
6.3.1.1. Type—automatic, manual, etc.
6.3.1.2. Condition of equipment
6.3.1.3. Batching sequence
6.3.2. Mixing
6.3.2.1. Type—central mix, truck mix, job mix, shrink mix, etc.
6.3.2.2. Condition of equipment
6.3.2.3. Mixing time
6.3.3. Method of transporting—trucks, buckets, chutes, pumps, etc.
6.3.4. Placing
6.3.4.1. Methods—conventional, underwater slipform, etc.
6.3.4.2. Equipment—buckets, elephant trunks, vibrators, etc.
6.3.4.3. Weather conditions—time of year, rain, snow, dry wind, temperature, humidity, etc.
6.3.4.4. Site conditions—cut, fill, presence of water, etc.
6.3.4.5. Construction joints
6.3.5. Finishing
6.3.5.1. Type—slabs, floors, pavements, appurtenances
6.3.5.2. Method—hand or machine
6.3.5.3. Equipment—screeds, floats, trowels, straight-edge, belt, etc.
6.3.5.4. Additives, hardeners, water, dust coat, coloring, etc.
6.3.6. Curing Procedures
6.3.6.1. Method—water, covering, curing compounds
6.3.6.2. Duration
6.3.6.3. Efficiency
6.3.7. Form removal (time of removal)

7. Initial physical properties of hardened concrete
7.1. Strength—compressive, flexural, elastic modulus
7.2. Density
7.3. Percentage and distribution of air
7.4. Volume change potential
7.4.1. Shrinkage or contraction
7.4.2. Expansion or swelling
7.4.3. Creep
7.5. Thermal properties

8. Additional items pertaining to pavements
8.1. Structural section (sketch and thickness of pavement layers—base, subbase, etc.)
8.2. Joints
8.2.1. Type, spacing, design
8.2.2. Condition
8.2.3. Filling material
8.2.4. Faulting—(measured in mm)
8.3. Cracks
8.3.1 Type (longitudinal, transverse, corner), size (measured in mm), frequency
8.4. Patching
8.5. Riding quality (as measured by instruments such as the BPR roughometer, the CHLOE profilometer, or profilograph present serviceability index, (PSI), etc.)
8.6. Condition of shoulders and ditches

APPENDIX

DEFINITION OF TERMS ASSOCIATED WITH THE DURABILITY OF CONCRETE

A.1 Cracks: An incomplete separation into one or more parts with or without space between.

A.1.1. Cracks will be classified by direction, width and depth. The following adjectives can be used: longitudinal, transverse, vertical, diagonal, and random. Three width ranges are suggested as follows: fine—generally less than 1 mm; medium—between 1 and 2 mm; wide—over 2 mm (see Fig. A.1.1.a through A.1.1.h).

A.1.2. *Pattern cracking*: Fine openings on concrete surfaces in the form of a pattern; resulting from a decrease in volume of the material near the surface, or increase in volume of the material below the surface, or both (see Fig. A.1.2.a through A.1.2.c).

A.1.3. *Checking*: Development of shallow cracks at closely spaced but irregular intervals on the surface of mortar or concrete (see Fig. A.1.3).

A.1.4. *Hairline cracking*: Small cracks of random pattern in an exposed concrete surface.

A.1.5. *D-cracking*: The progressive formation on a concrete surface of a series of fine cracks at rather close intervals, often of random patterns, but in highway slabs paralleling edges, joints, and cracks and usually curving across slab corners (see Fig. A.I.5.a and A.1.5.b).

A.2. Deterioration: Deterioration is any adverse change of normal mechanical, physical and chemical properties either on the surface or in the whole body of concrete generally through separation of its components.

A.2.1. *Disintegration*: Deterioration into small fragments or particles due to any cause (see Fig. A.2.1).

A.2.2. *Distortion*: Any abnormal deformation of concrete from its original shape (see Fig. A.2.2).

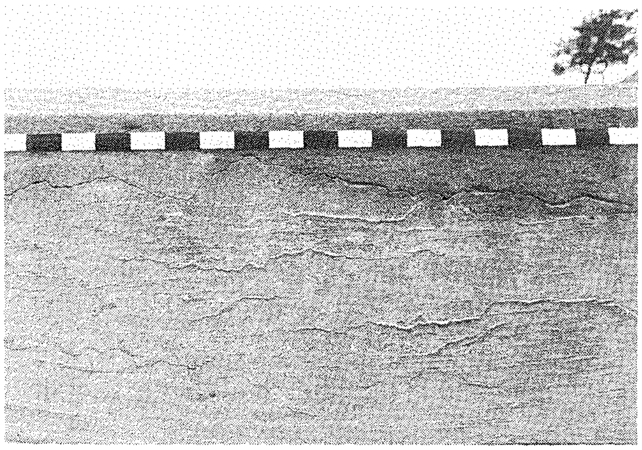

Fig. A.1.1.a—Longitudinal cracks (medium)

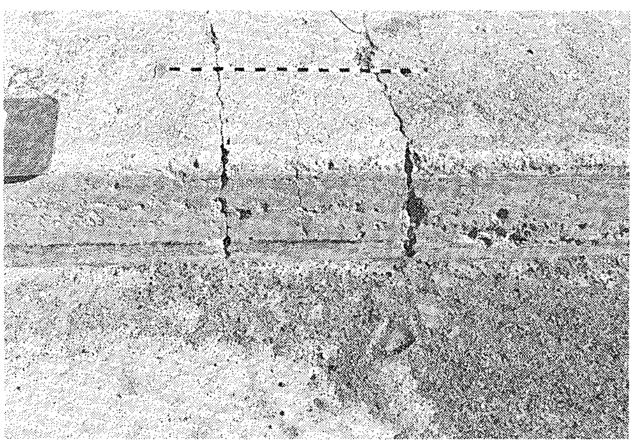

Fig. A.1.1.b—Transverse cracks (wide)

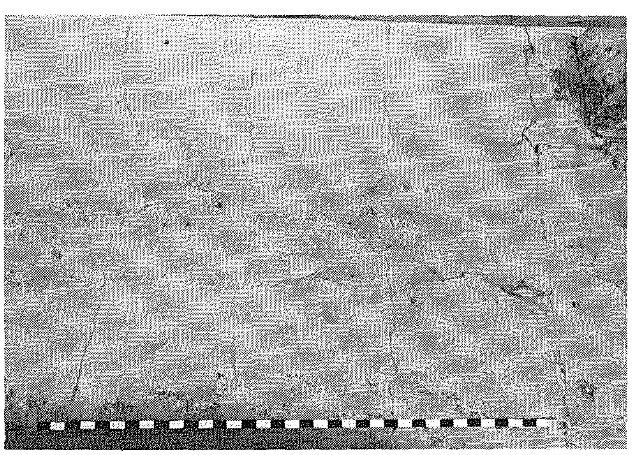

Fig. A.1.1.c—Transverse cracks (fine)

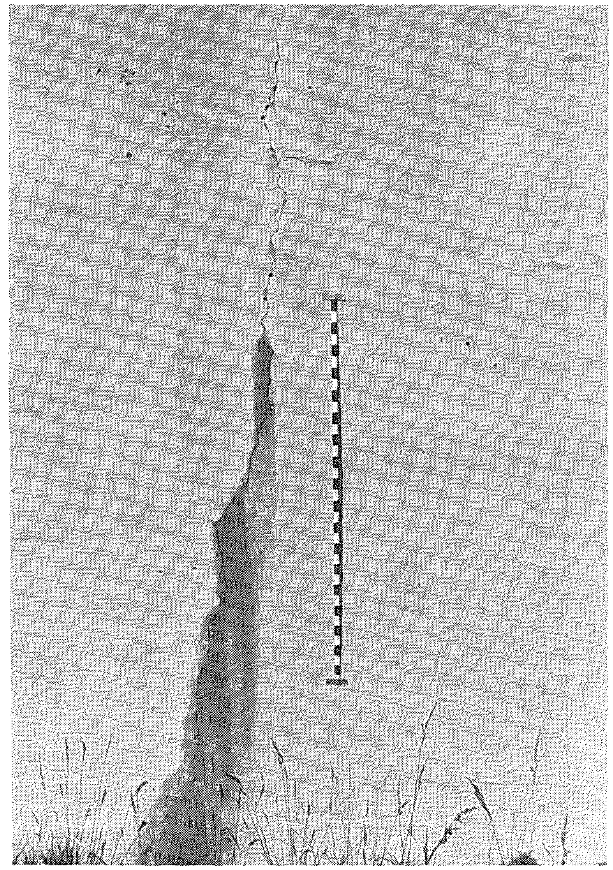

Fig. A.1.1.d—Vertical crack (medium)

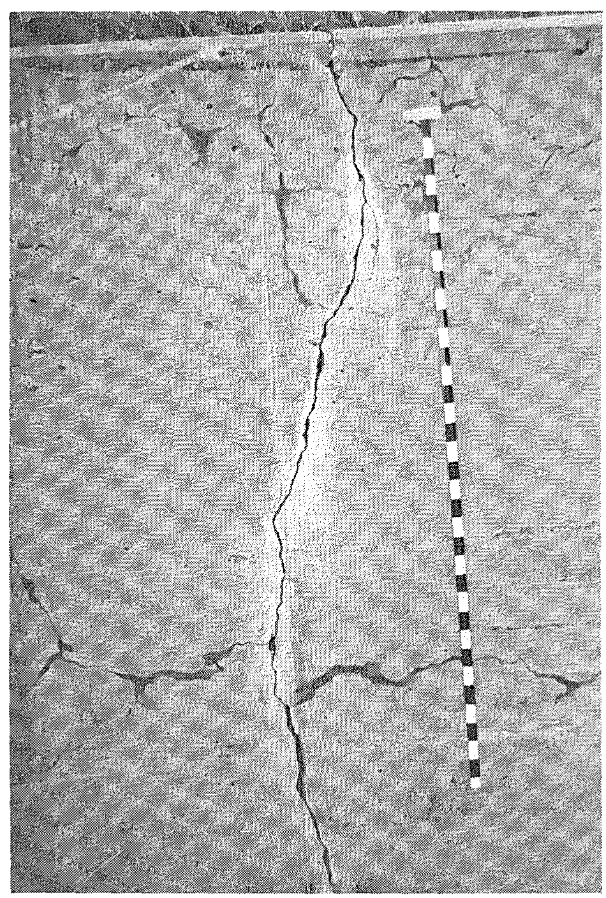

Fig. A.1.1.e—Vertical crack (wide)

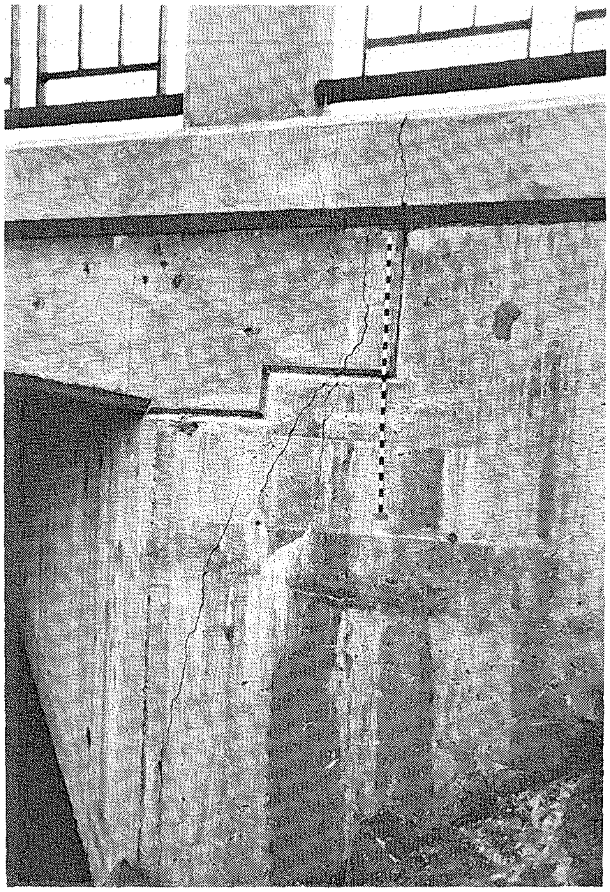

Fig. A.1.1.f—Diagonal cracks (wide)

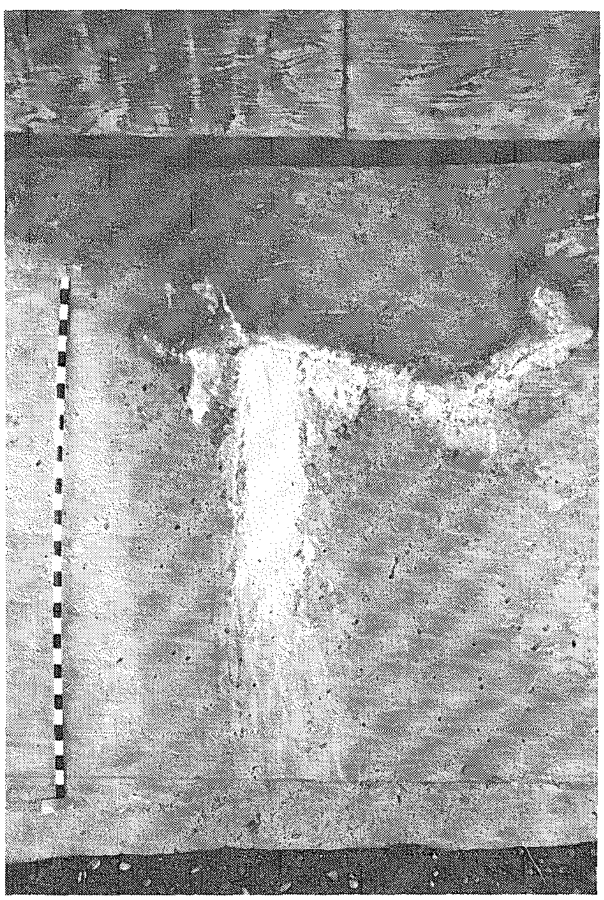

Fig. A.1.1.g—Random cracks (wide)

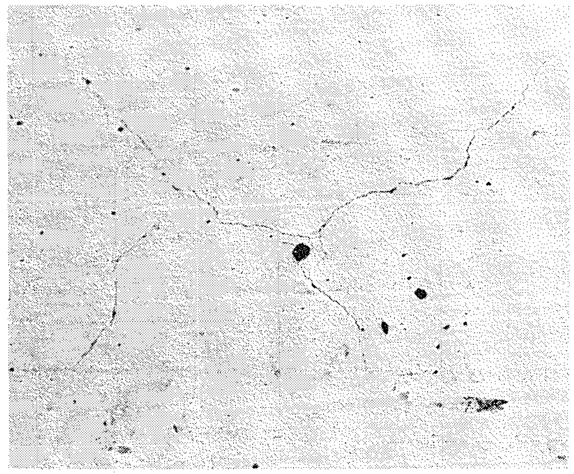

Fig. A.1.1.h—Random cracks (medium)

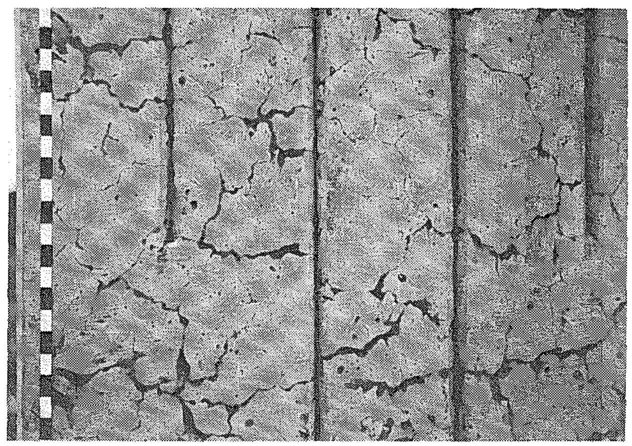

Fig. A.1.2.a—Pattern cracking (fine)

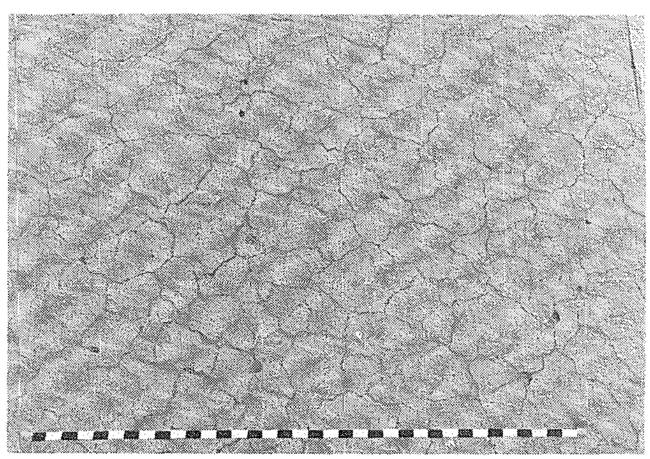

Fig. A.1.2.b—Pattern cracking (medium)

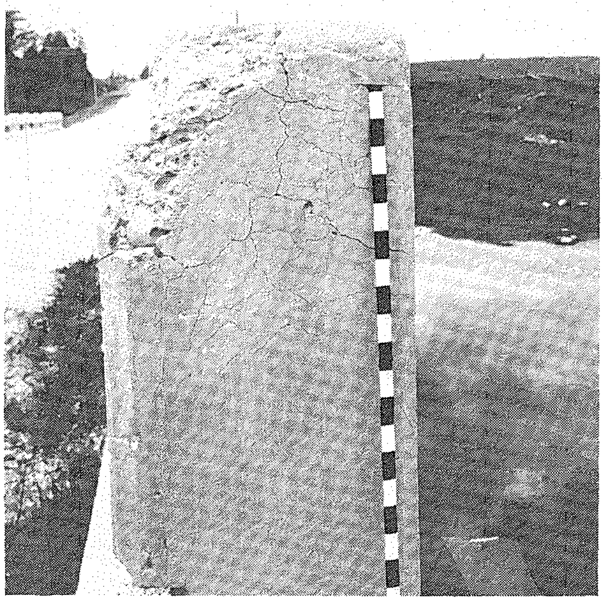

Fig.A.1.2.c—Pattern cracking (wide)

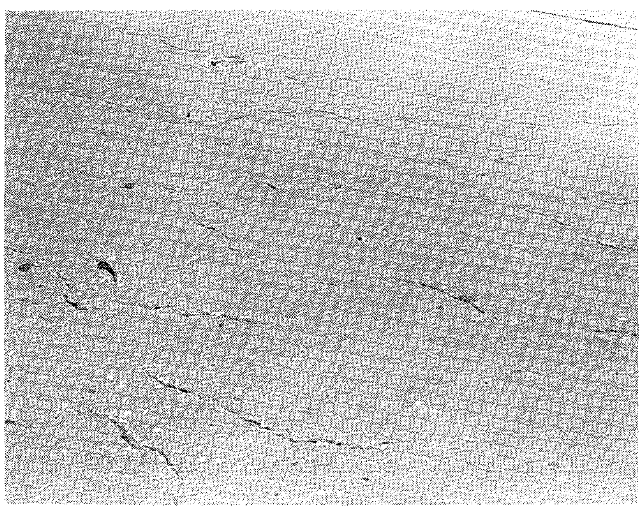

Fig. A.1.3—Checking (medium)

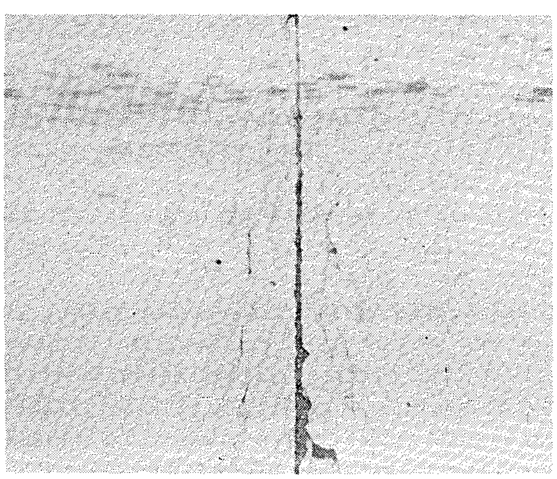

Fig. A.1.5.a—D-cracking (fine)

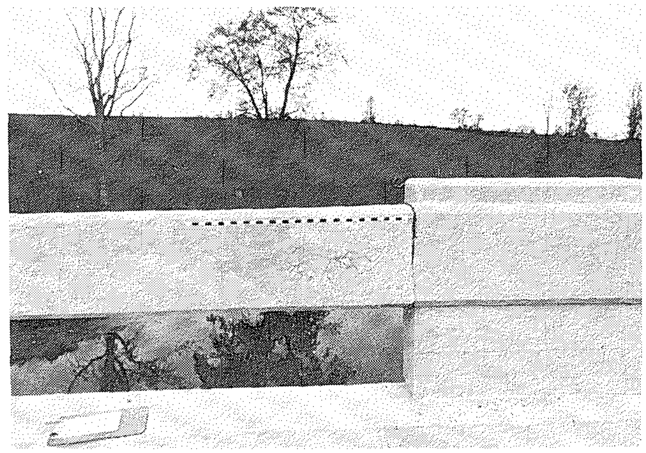

Fig. A.1.5.b—D-cracking (fine)

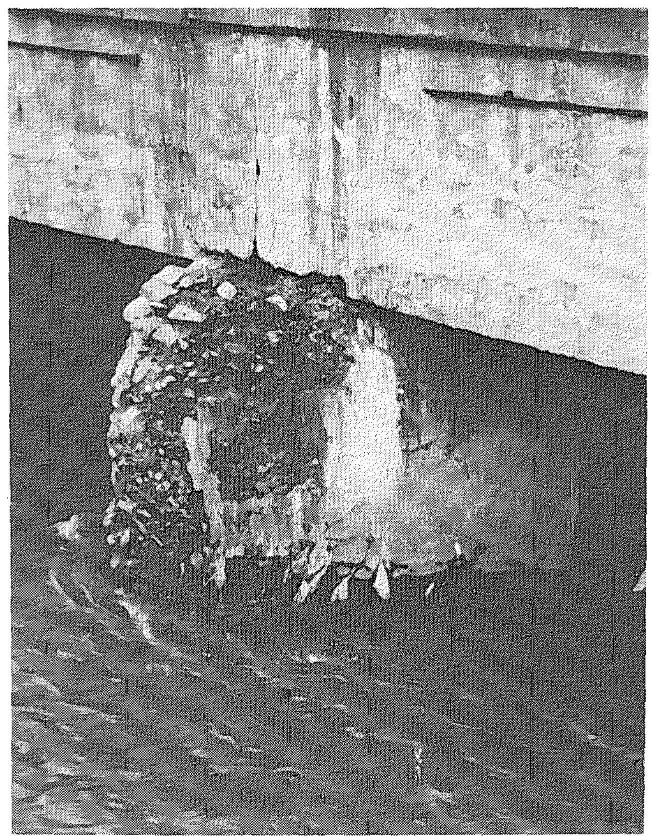

Fig. A.2.1—Disintegration

A.2.3. *Efflorescence*: A deposit of salts, usually white, formed on a surface, the substance having emerged from below the surface.

A.2.4. *Exudation*: A liquid or viscous gel-like material discharged through a pore, crack or opening in the surface (see Fig. A.2.4.a, A.2.4.b, and A.2.5).

A.2.5. *Incrustation*: A crust or coating generally hard formed on the surface of concrete or masonry construction (see Fig. A.2.5)

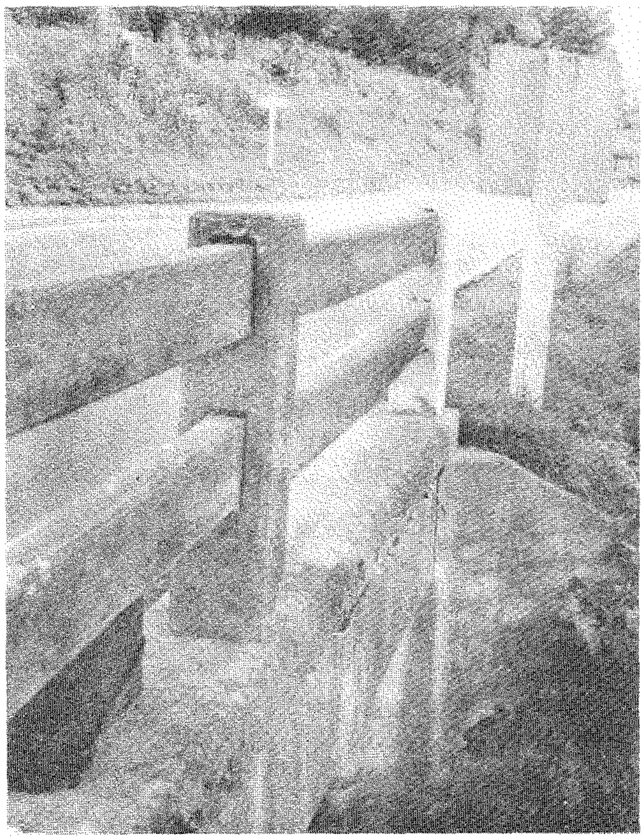

Fig. A.2.2—Distortion

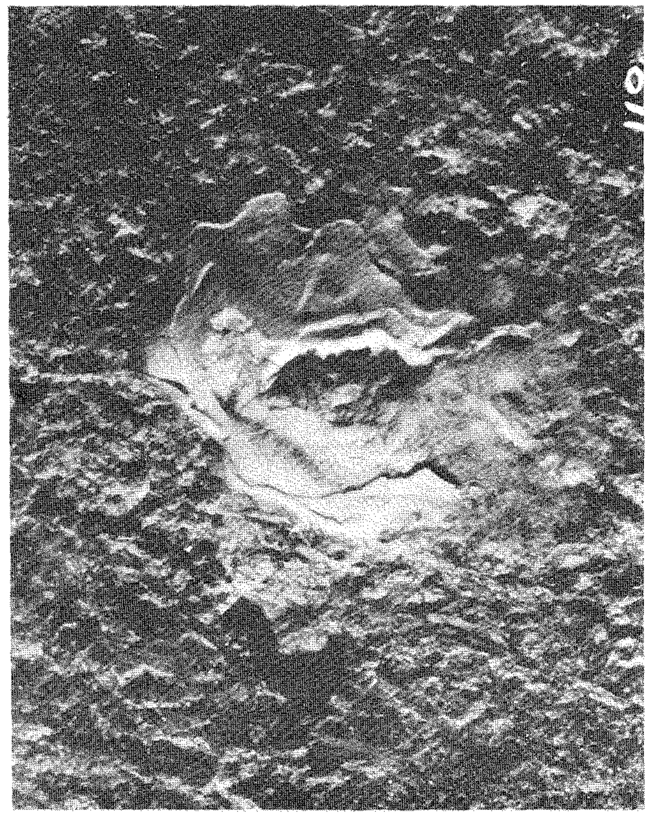

Fig. A.2.4.b—Exudation

Fig. A.2.4.a—Exudation

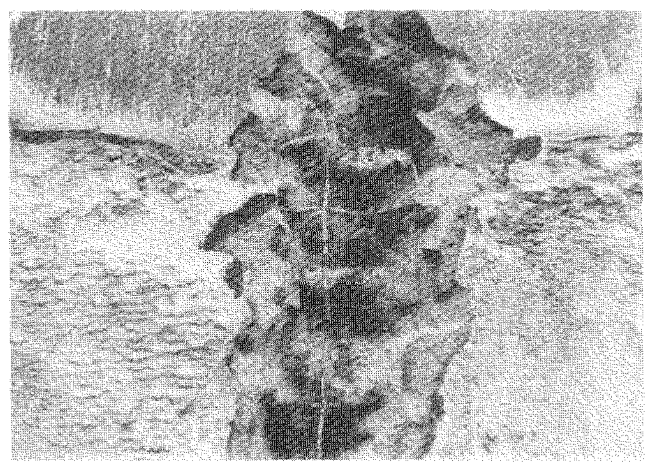

Fig. A.2.5—Exudation and incrustation

A.2.6. *Pitting*: Development of relatively small cavities in a surface, due to phenomena such as corrosion or cavitation, or, in concrete, localized disintegration.

A.2.7. *Popout*: The breaking away of small portions of a concrete surface due to internal pressure which leaves a shallow, typical conical, depression (see Fig. A.2.7).

A.2.7.1. ***Popouts, small:*** **Popouts leaving holes up to 10 mm in diameter, or the equivalent (see Fig.** A.2.7.1).

A.2.7.2. ***Popouts, medium:*** Popouts leaving **holes between 10 and 50 mm in diameter, or equi**valent (see Fig. A.2.7.2).

A.2.7.3. *Popouts, large:* Popouts leaving holes greater than 50 mm in diameter, or the equivalent (see Fig. A.2.7.3).

A.2.8. *Erosion*: Deterioration brought about by the abrasive action of fluids or solids in motion (see Fig. A.2.8).

A.2.9. *Scaling*: Local flaking or peeling away of the near surface portion of concrete or mortar.

A.2.9.1. *Peeling*: A process in which thin flakes of mortar are broken away from a concrete surface; such as by deterioration or by adherence of surface mortar to forms as forms are removed (see Fig. A.2.9.1.a and A.2.9.1.b).

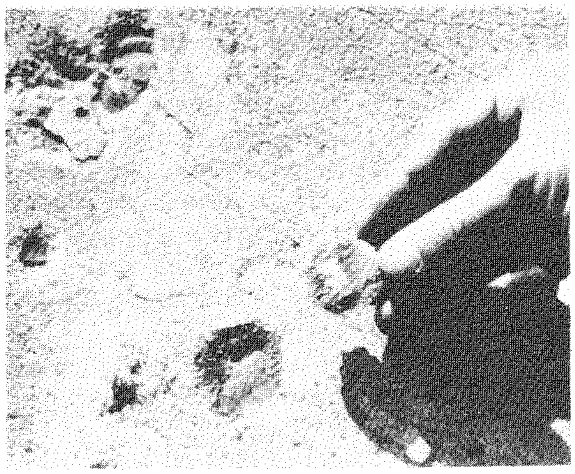

Fig. A.2.7—Popout

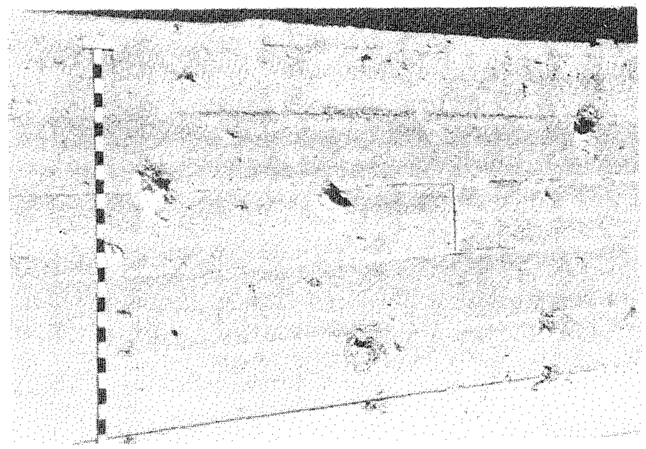

Fig. A.2.7.2—Popouts (medium)

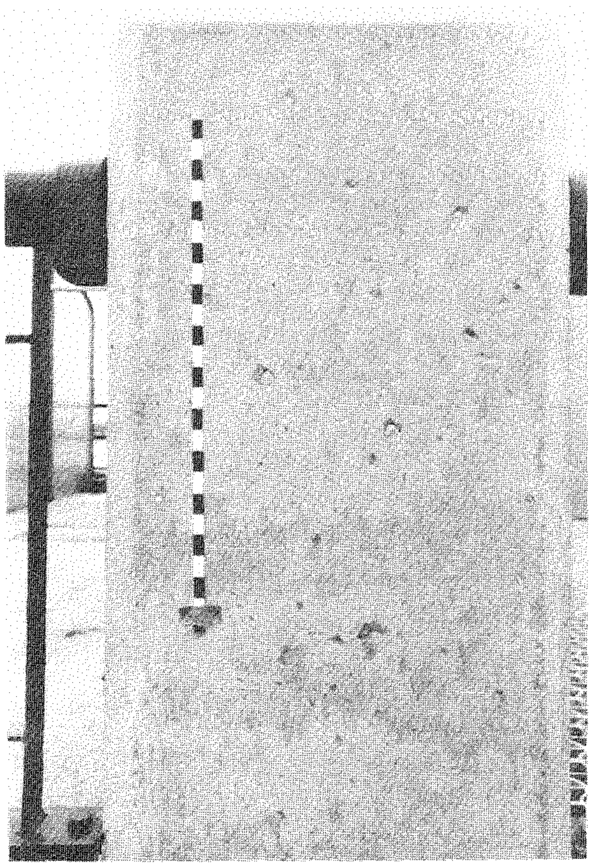

Fig. A.2.7.1—Popouts (small)

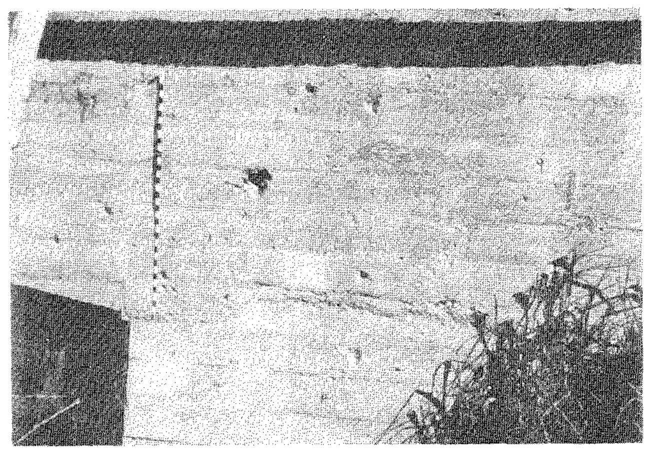

Fig. A.2.7.3—Popouts (large)

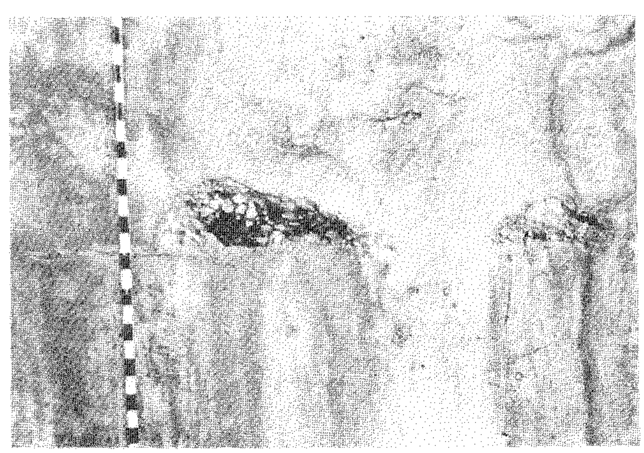

Fig. A.2.8—Erosion

A.2.9.2. *Scaling, light*: Loss of surface mortar without exposure of coarse aggregate (see Fig. A.2.9.2.a and A.2.9.2.b).

A.2.9.3. *Scaling, medium:* Loss of surface mortar up to 5 to 10 mm in depth and exposure of coarse aggregate (see Fig. A.2.9.3.a and A.2.9.3.b).

A.2.9.4. *Scaling, severe:* Loss of surface mortar 5 to 10 mm in depth with some loss of mortar surrounding aggregate particles 10 to 20 mm in depth, so that aggregate is clearly exposed and stands out from the concrete (see Fig. A.2.9.4.a and A.2.9.4.b).

A.2.9.5. *Scaling, very severe:* Loss of coarse aggregate particles as well as surface mortar and mortar surrounding aggregate, generally greater than 20 mm in depth (see Fig. A.2.9.5.a and A.2.9.5.b).

A.2.10. *Spall*: A fragment, usually in the shape of a flake, detached from a larger mass by a blow, by the action of weather, by pressure, or by expansion within the large mass.

A.2.10.1. *Small spall:* A roughly circular or oval depression generally not greater than 20 mm in depth nor greater than about 150 mm in any dimension, caused by the separation of a portion of the surface concrete (see Fig. A.2.10.1).

A.2.10.2. *Large spall:* May be roughly circular or oval depression, or in some cases an elongated depression over a reinforcing bar, generally 20 mm or more in depth and 150 mm or greater in any dimension, caused by a separation of the surface concrete (see Fig. A.2.10.2).

A.2.11. *Joint spall*: Elongated cavity along a joint (see Fig. A.2.11.a and A.2.11.b).

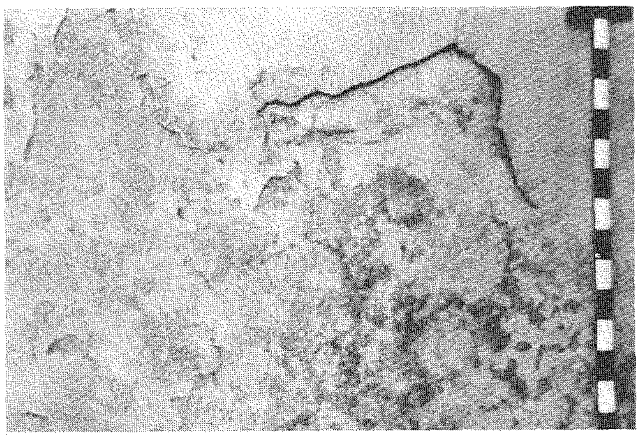

Fig. A.2.9.1.a—Close-up of peeling

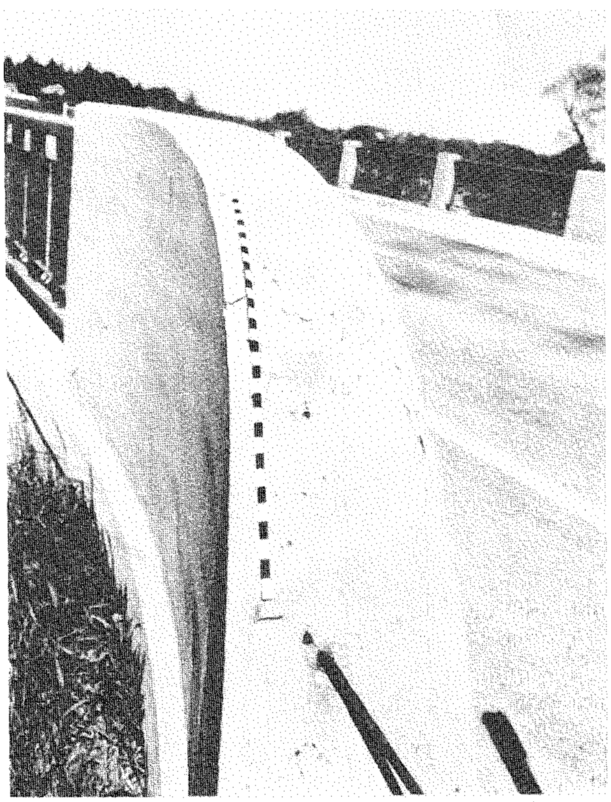

Fig. A.2.9.1.b—Peeling on bridge abutment

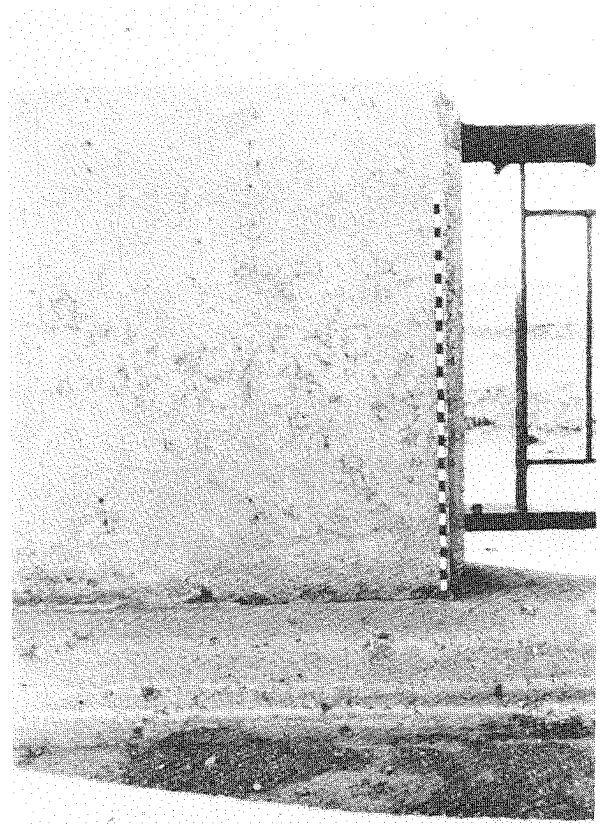

Fig. A.2.9.2.a—Scaling (light)

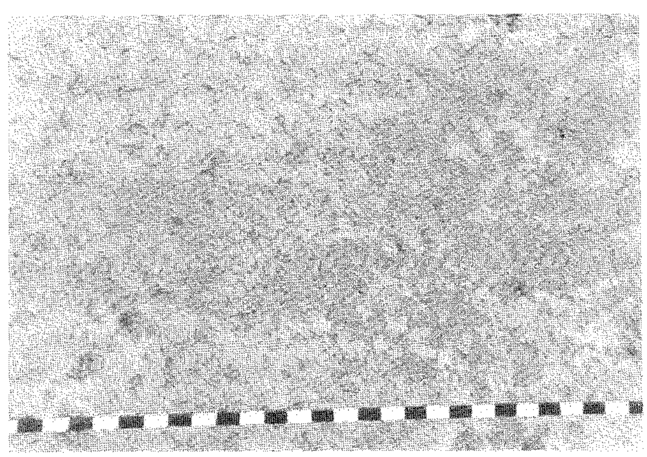

Fig. A.2.9.2.b—Close-up of scaling (light)

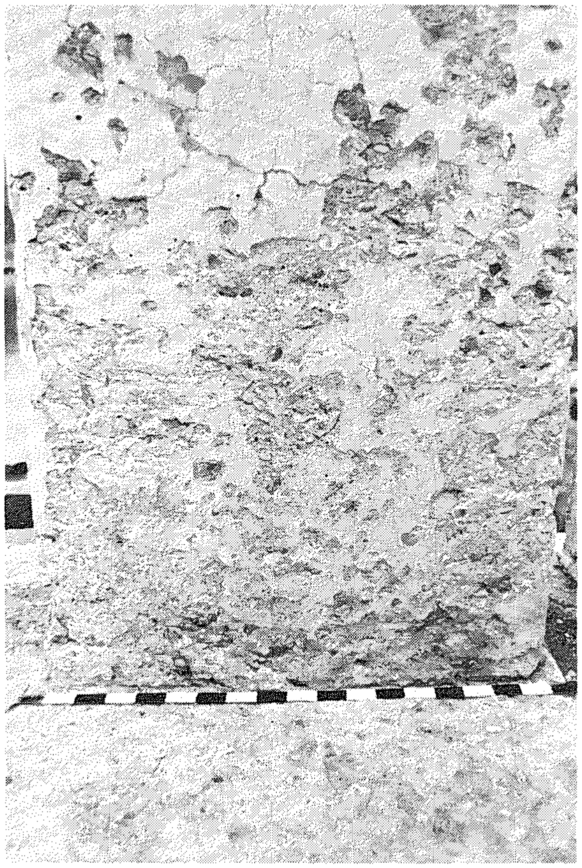

Fig. A.2.9.3.a—Scaling (medium)

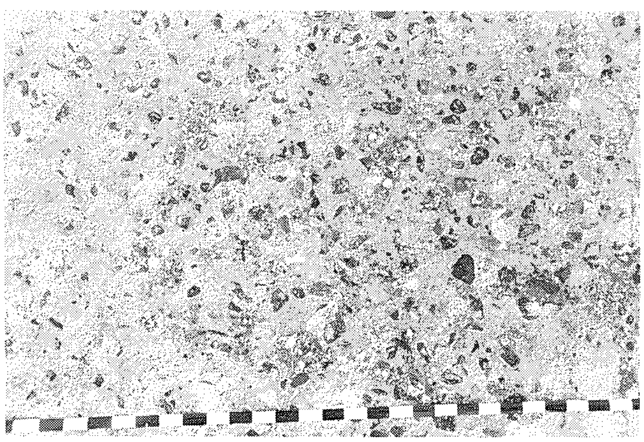

Fig. A.2.9.3.b—Close-up of scaling (medium)

Fig. A.2.9.4.a—Close-up of scaling (severe)

A.2.12. *Dummy area*: Area of concrete surface which gives off a hollow sound when struck.

A.2.13. *Stalactite*: A downward pointing formation, hanging from the surface of concrete, shaped like an icicle.

A.2.14. *Stalagmite*: As stalactite, but upward formation.

A.2.15. *Dusting*: The development of a powdered material at the surface of hardened concrete (see Fig. A.2.15).

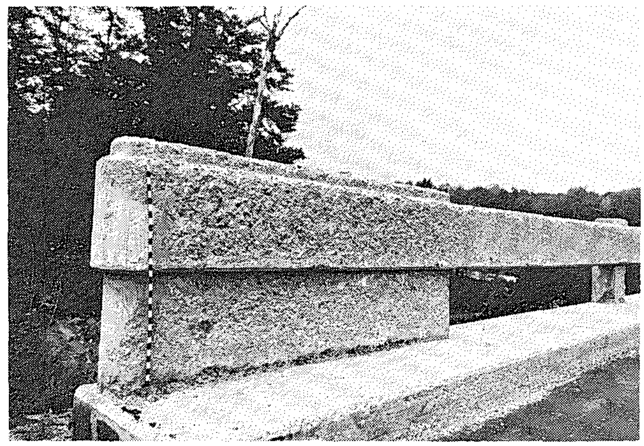

Fig. A.2.9.4.b—Scaling severe

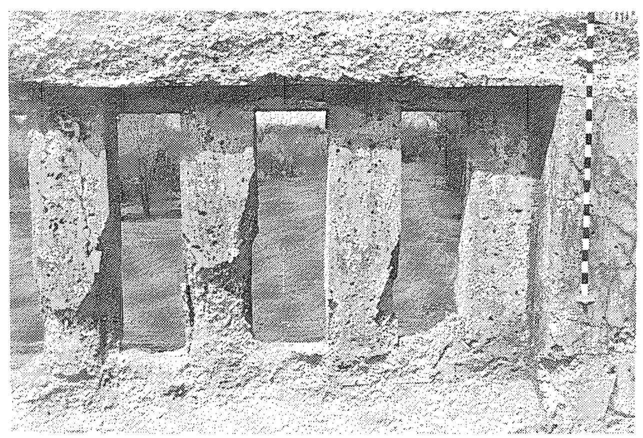

Fig. A.2.9.5.a—Scaling (very severe)

Fig. A.2.9.5.b—Close-up of scaling (very severe)

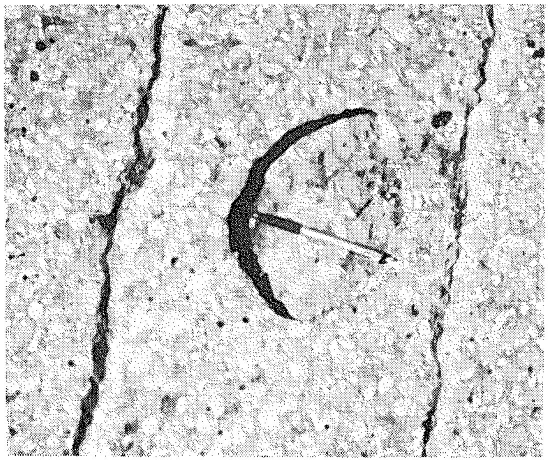

Fig. A.2.10.1—Small spall

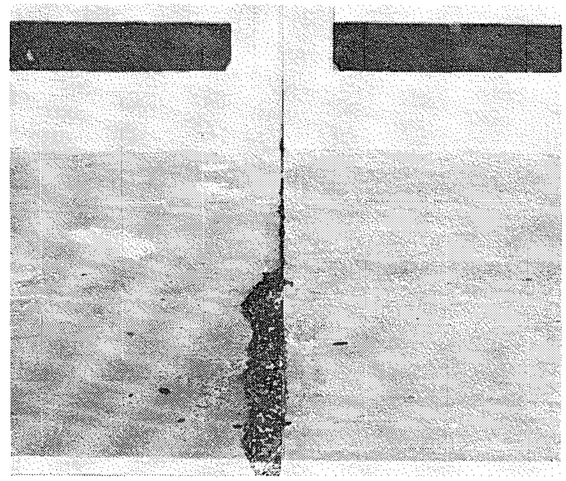

Fig. A.2.11.a—Joint spall

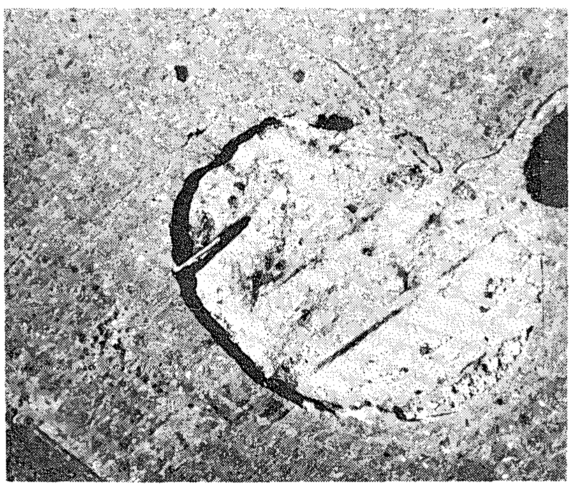

Fig. A.2.10.2—Large spall

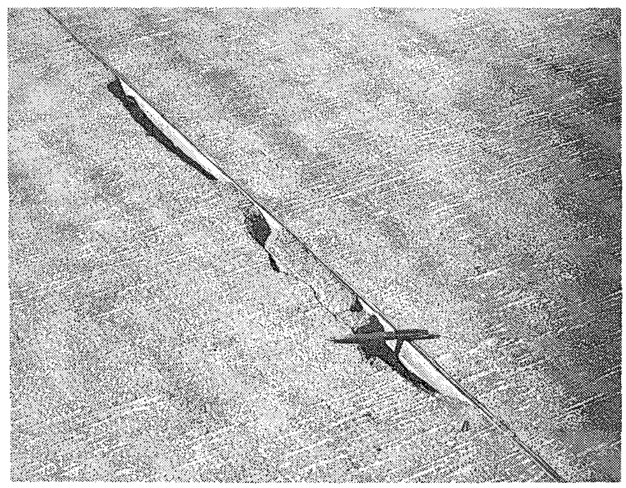

Fig. A.2.11.b—Joint spall

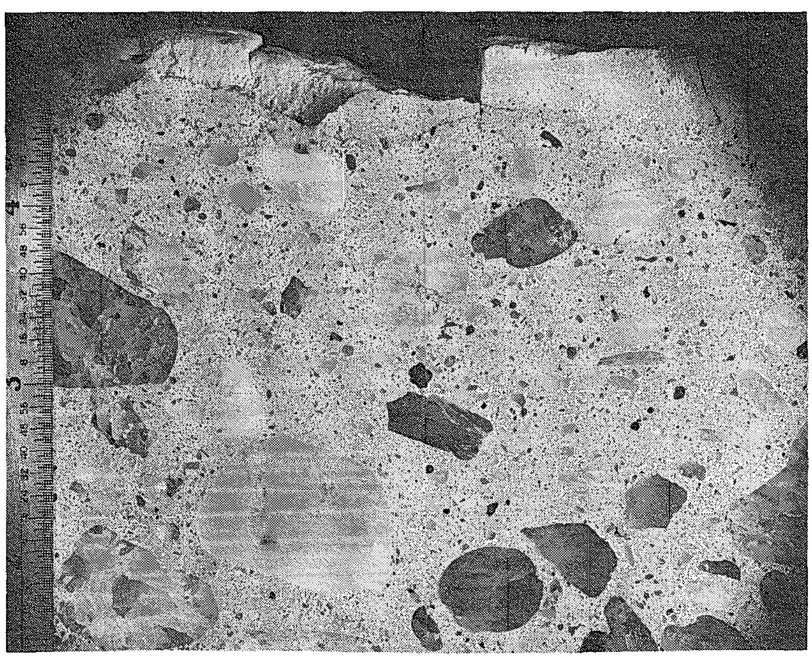

Fig. A.2.15—Dusting; surface at top of ruler is a floor surface of concrete placed *very* wet and which also carbonated; segregation is also evident

A.2.16. *Corrosion*: Disintegration or deterioration of concrete or reinforcement by electrolysis or by chemical attack (see Fig. A.2.16).

A.3. Textural defects:

A.3.1. *Bleeding channels*: Essentially vertical localized open channels caused by heavy bleeding (see Fig. A.3.1).

A.3.2. *Sand Streak*: Streak in surface of formed concrete caused by bleeding (see Fig. A.3.2).

A.3.3. *Water pocket*: Voids along the underside of aggregate particles or reinforcing steel which formed during the bleeding period. Initially filled with bleeding water.

A.3.4. *Stratification*: The separation of overwet or overvibrated concrete into horizontal layers with increasingly lighter material toward the top; water, laitance, mortar, and coarse aggregate will tend to occupy successively lower positions in that order; a layered structure in concrete resulting from placing of successive batches that differ in appearance (see Fig. A.3.4.).

A.3.5. *Honeycomb*: Voids left in concrete due to failure of the mortar to effectively fill the spaces among coarse aggregate particles (see Fig. A.3.5.a and A.3.5.b).

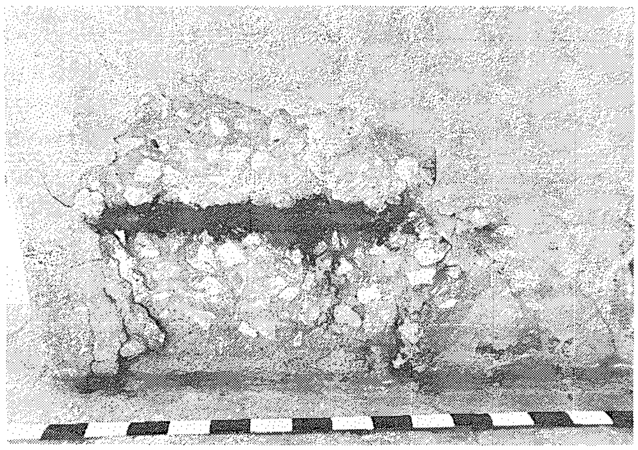

Fig. A.2.16—Corrosion

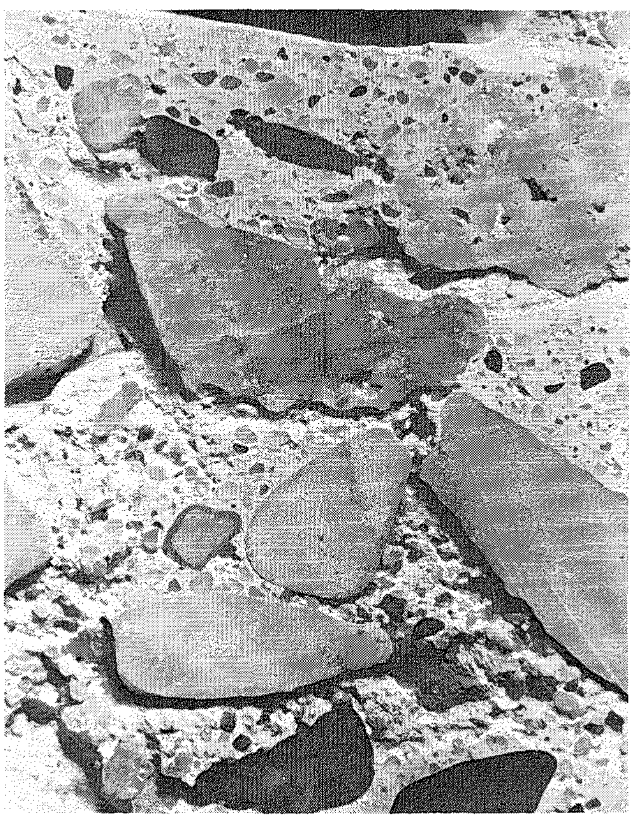

Fig. A.3.1—Bleeding channels and water pockets of concrete in a caisson; note laitance below particles of coarse aggregate

Fig. A.3.2—Sand streaking on a vertical formed surface

Fig. A.3.4—Stratification

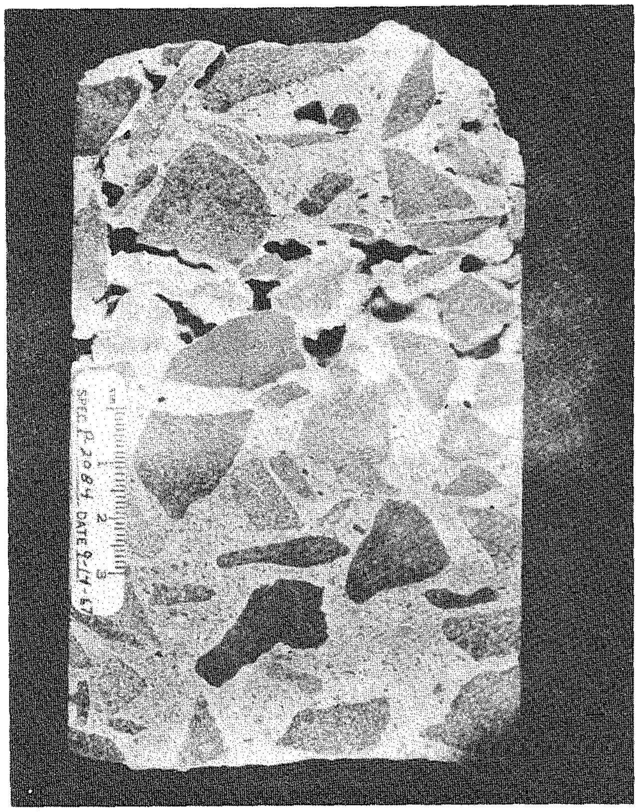

Fig. A.3.5.a—Honeycomb

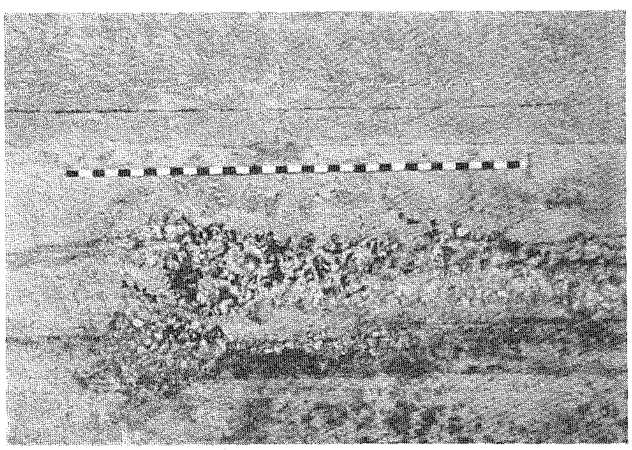

Fig. A.3.5.b—Honeycomb

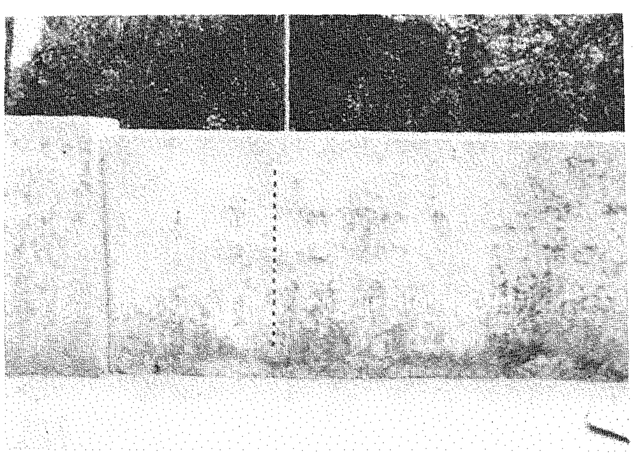

Fig. A.3.8—Discoloration

A.3.6. *Sand Pocket*: Part of concrete containing sand without cement.

A.3.7. *Segregation*: The differential concentration of the components of mixed concrete, resulting in non uniform proportions in the mass.

A.3.8. *Discoloration*: Departure of color from that which is normal or desired (see Fig. A.3.8).

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3. *Trilingual Dictionary of Engineering Materials Testing,* RILEM Bulletins 20-25, Paris, 1955.

This report was approved by letter ballot of the committee and reported to ACI headquarters Jan. 5, 1967. At the time of balloting (late 1966), the committee consisted of 22 members, of whom 19 voted affirmatively, 1 negatively, one "conditionally" affirmative, and one not returning his ballot.

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ACI 207.1R-70

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Mass Concrete for Dams and Other Massive Structures

Reported by ACI Committee 207

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This report presents a discussion of the materials and practices employed in proportioning, mixing, placing, and curing mass concrete, and of the properties and behavior of the hardened mass concrete. Particular emphasis is placed on the differences between mass concrete and conventional concrete. It is designed to serve as a reference for those engaged in the design and construction of concrete dams and other massive concrete structures.

Keywords: admixtures; aggregate gradation; aggregate size; aggregates; air entrainment; arch dams; batching; bridge piers; cements; compacting; compressive strength; concrete construction; concrete dams; concrete durability; cooling; cracking (fracturing); creep properties; curing; diffusivity; formwork (construction); heat of hydration; history; mass concrete; measuring instruments; mix proportioning; mixing; modulus of elasticity; permeability; placing; Poisson's ratio; pozzolans; shear properties; shrinkage; stresses; temperature control; temperature rise (in concrete); thermal expansion; thermal gradient; thermal properties; vibration; volume change.

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CHAPTER 1 — INTRODUCTION

1.1 — Scope

This report presents a detailed discussion of the materials and practices employed in proportioning, mixing, placing, and curing mass concrete, and of the properties and behavior of the hardened mass concrete. Particular emphasis is placed on mass concrete used in the construction of concrete gravity and arch type dams and other *nonreinforced* structures usually incorporating large size coarse aggregate. The effects of heat generation and volume changes on the design and behavior of massive *reinforced* elements and structures are not discussed but will be covered in a future report.

1.2 — Definition

Mass concrete is herein defined as:

> "Any large volume of cast-in-place concrete with dimensions large enough to require that measures be taken to cope with the generation of heat and attendant volume change to minimize cracking."

Mass concrete structures, such as dams, resist loads by virtue of their structural shape, mass, and strength.

1.3 — History

1.3.1 *General*—When concrete was first used in small dams it was mixed by hand, the portland cement usually had to be "aged" to stand the boiling test, the aggregate was bank-run sand and gravel, and proportioning was by the shovelful. Tremendous progress has been made, and the art and science of dam building practiced by the principal agencies engaged in such work has reached a high degree of perfection and control. The selection and proportioning of concrete materials to produce suitable properties of the finished product (strength, durability, deformability, impermeability) can be predicted and controlled with accuracy.

This report covers in chronological order the principal steps from those very small beginnings to the present, where on large dam constructions there is exact and automatic proportioning and mixing of materials, and concrete in 12-cu yd (9-m^3) buckets is placed at the rate of 10,000 cu yd (7650 m^3) per day at a temperature of less than 50° F (10 C) as placed during the hottest weather. Lean mixes are now made workable by such means as air entrainment and the use of finely divided pozzolanic materials. Water-reducing and set-controlling admixtures are effective in reducing the required cement content to a minimum as well as in controlling the time of setting. Those concerned with concrete dam construction certainly should not feel that the ultimate has been reached, but they are certainly justified in feeling some satisfaction with the progress that has been made.

1.3.2 *Prior to 1900*—Prior to the beginning of the century, much of the portland cement used in the United States was imported from Europe. All cements were very coarse by present standards and quite commonly they were underburned, requiring a long period of storage to develop soundness. For dams of that period, aggregates used were usually bank-run sand and gravel, employed without benefit of washing to remove objectionable dirt and fines. As a result, concrete mixes varied widely in cement content and in sand-aggregate ratio. Concrete mixing was usually by hand and proportioning was by shovel, wheelbarrow, box, or cart. The effect of water-cement ratio was unknown and generally no attempt was made to control the volume of mixing water. There was no measure of consistency except by visual observation of the mix.

Some of the dams were of cyclopean masonry in which plums (large stones) were partially embedded in a very wet concrete. The spaces between plums were then filled with concrete, also very wet. Some of the early dams were built without contraction joints and without regular lifts. However, there were notable exceptions where concrete was cast in blocks; the height of lift was regulated and concrete of very dry consistency was placed in thin layers and consolidated by rigorous hand tamping.

Generally, mixed concrete was transported to the forms by wheelbarrow, although where plums were employed in cyclopean masonry, stiff-leg derricks erected inside the work moved the wet concrete and plums. The maximum rate of placement was only a few hundred cubic yards a day. Generally, there was no attempt at moist curing.

One very notable exception to these general practices was Crystal Springs Dam constructed in 1888, which is located near San Mateo, Calif., about 20 miles south of San Francisco. According to available information, this was the first dam in the United States in which the maximum permissible quantity of mixing water was specified. The concrete for this 154 ft high structure was cast in a system of interlocking blocks of specified shape and dimensions. Fresh concrete was covered with planks as a protection from the sun and was kept wet until hardening had taken place. For this very dry concrete, an old photograph indicates that five hand tampers were employed for each man with a wheelbarrow.

Only a few of the concrete dams built in the United States prior to 1900 remain serviceable today, and most of them are small. Of the 2800 listings in the 1958 *Register of Dams in the United States*[1] there appeared to be no more than 18 that were built prior to 1900. More than a third of these are in the states of California and Arizona where the climate is mild. Of those in the more rigorous climate of the East and Middle West, except where they were faced with stone masonry, time and weather have taken a considerable toll.

1.3.3 *Years 1900 to 1930*—After the turn of the century, between 1900 and about 1930, the construction of all types of concrete dams was greatly accelerated. More and higher dams for irrigation, power, and water supply were the order of the day. Concrete placement by means of towers and chutes became the vogue. In the United States, the portland cement industry became well established, and cement was rarely imported from Europe. ASTM specifications for portland cement underwent little change during the first 30 years of this century aside from a modest increase in fineness requirement determined by sieve analysis. Except for the limits on magnesia and loss on ignition, there were no chemical requirements.

However, much more attention was being given to the character and grading of aggregates and very substantial progress was made in the development of methods of proportioning concrete. In general, little attention was paid to quantity of mixing water, and its effect on the strength of concrete was largely unknown until the publication of the results of investigations made by Duff Abrams and his associates during the period 1916-1926. The use of excessively wet mixes which could be easily chuted down flat slopes was the rule rather than the exception for several years even after the water-cement ratio law had become well established.

Generally, portland cements were employed without admixture of any kind. Exceptions were the sand-cements employed by the U.S. Reclamation Service, now the U.S. Bureau of Reclamation, in the construction of Elephant Butte and Arrowrock dams. The latter, 350 ft (107 m) high, at the time of its completion in 1915, was the highest dam in the world. The interior mass concrete of this gravity arch dam contained only about 376 lb of sand-cement per cu yd (223 kg/m^3). It was produced on the job, intergrinding about equal parts of portland cement and pulverized granite to a fineness such that not less than 90 percent passed the 200-mesh sieve. As compared with portland cements then on the market, the sand-cement was of unusually high fineness.

Another exception was one of the abutments of Big Dalton Dam, a multiple arch dam built by the Los Angeles Flood Control District during the late twenties. Pumicite (a pozzolan) from Friant, Calif., was employed as a 20-percent replacement by weight for portland cement.

During the 1900-1930 period, cyclopean concrete went out of style. For dams of thick section, the maximum size of aggregate for mass concrete was increased to as large as 10 in. (25 cm). As a means of measuring consistency, the slump test had come into use. The testing of 6 x 12-in. (15 x 30 cm); and 8 x 16-in. (20 x 40-cm) job cylinders became common practice in the United States while European countries generally adopted the 8 x 8-in. (20 x 20-cm) cubes for testing the strength at various ages. Mixers of 3 cu yd (2.3 cm^3) capacity were in common use near the end of this period and there were some of 4 cu yd (3 cm^3) capacity.

In the eastern parts of the United States where freezing and thawing conditions were severe, it was not uncommon to employ as much as 564 lb of cement per cu yd (336 kg/m^3) for the entire concrete mass. In the West and other areas of mild climate, portland cement content as low as 376 lb per cu yd (223 kg/m^3) was employed, and the practice of using a richer mix such as 564 lb per cu yd for the exterior concrete exposed to weather with 376 lb mix for the interior had come into use in areas where the climate was severe.

An exception was Roosevelt Dam built during 1905-1911. It is a rubble masonry structure faced with rough ashlar blocks laid in portland cement mortar using a cement manufactured in a plant near the dam site. For this structure the average cement content has been calculated to be approximately 282 lb per cu yd (168 kg/m^3). For the interior of the mass, rough quarry stones were embedded in a 1:2.5 mortar containing about 846 lb of cement per cu yd (500 kg/m^3). The voids between the closely spaced stones in each layer were filled with a concrete containing 564 lb of cement per cu yd (336 kg/m^3) into which spalls were spaded by hand. These conditions account for the very low average cement content. The rate of construction was laboriously slow, and Roosevelt Dam represents perhaps the last of the large dams so built in the United States.

1.3.4 *Years 1930 to 1965*—By the end of 1929, although sloppy concrete placed by chuting was often employed [frequently with 3 in. (7.6 cm) maximum size aggregate and generally of high water-cement ratio], on the larger and more closely controlled constructions, cement and carefully processed aggregates were proportioned by weight, and mixing water by volume. Slumps as low as 3 in. were employed without vibration, and concrete was being transported at least on one job from mixer to form in 8 cu yd (6-cm^3) buckets. This was an era of rapid development in mass concrete construction for dams.

From a study of the records and actual inspection of a considerable number of dams, it appears that there were differences in condition which could not be explained. Of two structures that appeared to be of like quality subjected to the same environment, one might exhibit excessive cracking and other signs of distress while the other, after a like period of service, appeared to be in perfect condition. Meager records of a few dams indicated wide variation in temperature due to hydration of cement and that the degree of cracking was associated with the temperature rise. For the purpose of learning more about the significant properties of mass concrete in dams and factors which influence these properties, ACI Committee 207, Mass Concrete, was organized in 1930 (originally as Committee 108). Bogue and his associates under the PCA fellowship at the National Bureau of Standards had already identified the principal compounds in portland cement, and Hubert Woods and associates were engaged in investigations to determine the contributions of these compounds to heat of hydration and to strength of mortars and concretes.

By the beginning of 1930, Hoover Dam was in early prospect. Because of the unprecedented size of the structure, investigations much more elaborate than any that had been previously undertaken, were carried out to determine effect of composition and fineness of cement, cement factor, temperature of curing, maximum size of aggregate, etc., on heat of hydration of cement, compressive strength, and other properties of mortars and concrete.

The results of these investigations led to the use of low-heat cement in Hoover Dam. The investigations also furnished information for the design of the embedded pipe cooling system employed for the first time in Hoover Dam. Low-heat cement was first used in Morris Dam, near Pasadena, Calif., which was started a year before Hoover Dam.

For Hoover Dam, the construction plant was of unprecedented capacity. Batching and mixing were completely automatic. The record days' output for the two concrete plants [equipped with 4-cu yd (3 m^3) mixers] was over 10,000 cu yd (7600 m^3). Concrete was transported in 8 cu yd (6 m^3) buckets by cableways and compacted initially by ramming and tamping. In the spring of 1933, large internal vibrators were introduced and were used extensively for compacting the remainder of the concrete. Within about 2 years, 3,200,000 cu yd (2,440,000 m^3) of concrete were placed.

Hoover Dam marked the beginning of an era of improved practices in large concrete dam construction. Completed in 1935 at a rate of construction then unprecedented, the practices there employed, with some refinements, have been in use on most of the large concrete dams which have been constructed in the United States and in many other countries all over the world since that time.

During the late twenties and the early thirties, it was practically the unwritten law that no mass concrete for large dams should contain less than 376 lb of cement per cu yd (223 kg/m^3) and some of the authorities of that period were of the opinion that the cement factor should never be less than 564 lb per cu yd (336 kg/m^3). For Norris Dam, which was completed by the Tennessee Valley Authority in 1936 and for which the cement factor of the interior of the dam was 376 lb per cu yd the degree of cracking was objectionably great although the strength of wet-screened 6 x 12-in. (15 x 30-cm) job cylinders at the age of 1 year was nearly 7000 psi (490 kg/cm^2). Later, 18 x 36-in. (45 x 90-cm) cores of 376 lb per cu yd concrete cut from the first stage of construction of Grand Coulee Dam at the age of 2 years exhibited strengths in excess of 8000 psi (560 kg/cm^2). Judged by composition, the cement was of the moderate heat type corresponding to the present Type II. Considering the magnitude of the calculated stresses within the structure, it was evident that such high compressive strengths were quite unnecessary and that a reduction in cement content on similar future constructions might be expected to substantially reduce the tendency toward cracking.

For Hiwassee Dam, completed by TVA in 1940, the 376-lb (223 kg/m^3) barrier was broken. For that structure the cement content of the mass concrete was only 282 lb per cu yd (168 kg/m^3), an unusually low value for that time. Hiwassee Dam was singularly free from cracks, and there began a trend toward reducing the cement content which is still continuing. Because of low concrete strengths at early ages, the use of low-heat portland cement in the construction of dams has been almost entirely discontinued for large gravity dams, notably those built by the Corps of Engineers. The Type II cement content of the interior mass concrete has been in the order of 235 lb per cu yd (140 kg/m^3) and even as low as 212 lb per cu yd (126 kg/m^3). An example of a large gravity dam for which the Type II cement content for mass concrete was 235 lb per cu yd is Pine Flat in California, completed by the Corps of Engineers in 1954. In high dams of the arch type where stresses are moderately high, the cement content of the mass mix is usually in the range of 300 to 450 lb per cu yd (180 to 270 kg/m^3), the higher cement content being used in the thinner and more highly stressed dams of this type.

Examples of cement contents for recent high arch dams are:

(1) 282 lb per cu yd (168 kg/m^3) of cement and pozzolan in Glen Canyon Dam, a relatively thick arch dam in Arizona

(2) 385 lb per cu yd (230 kg/m^3) of cement in Morrow Point Dam in Colorado

(3) 420 lb per cu yd (250 kg/m^3) of cement in El Etazar Dam near Madrid, Spain

1.3.5 *Precooling*—To reduce the maximum temperature of mass concrete during the hydration period, the practice of precooling concrete materials prior to mixing was started in the early forties and was extensively employed in the construction of large dams during the late forties and fifties. This practice avoids not only the building of high internal stresses, due to the different temperature drops of the concrete in the interior and near the surface, but also the autogenous expansion of the concrete which takes place in many cases due to the presence of lime and magnesium oxide. This expansion, when present, increases with temperature. This phenomenon has been observed from data obtained from "no-stress" strain meters embedded in the concrete.

The first serious effort undertaken toward precooling appears to have been in the construction of Norfolk Dam during 1941-1945 by the Corps of Engineers. During the warmer months the plan was to introduce crushed ice into the mixing water. By so doing, the temperature of freshly mixed mass concrete could be reduced by about 10 F (5.5 C). On later jobs not only has crushed ice been used in the mixing water, but coarse aggregates have been precooled either by cold air or cold water in the batching plant. Recently, both fine and coarse aggregates in a moist condition have been precooled by various means including vacuum. It has become almost standard practice in the United States to employ precooling for large dams in regions where the summer temperatures are high to assure that the temperature of fresh concrete as it is deposited in the work does not exceed about 50 F (10 C).

On some large dams, a combination of precooling as just described and postcooling by embedded pipe refrigeration, as at Hoover Dam, has been used. A good example of this practice is Glen Canyon Dam where at times during the summer months the ambient temperatures were considerably greater than 100 F (38 C). The temperature of the precooled fresh concrete did not exceed 50 F (10 C). By means of embedded pipe refrigeration, the maximum temperature of hardening concrete has been kept below 75 F (24 C).

1.3.6 *Strength requirements*—Another interesting development of the fifties has been the abandonment of the 28-day strength as a design requirement. Maximum stresses under load do not usually develop until the concrete is at the age of 1 year or more. Under mass curing conditions, with the cement and pozzolans customarily employed, the gain in concrete strength between 28 days and 1 year is generally large varying from 30 percent to more than 200 percent depending on the quantities and proportioning of cementitious materials and properties of the aggregates. It has become the practice of some designers of dams to specify the desired strength of mass concrete in 18 x 36-in. (45 x 90-cm) test cylinders at the age of 1 year or even in some cases at the age of 2 years. For routine quality control in the field 6 x 12-in. (15 x 30-cm) cylinders are normally used with all aggregate larger than 1½ in. (4 cm) removed by screening of the wet concrete. Strength requirements of the wet screened concrete at 28 days are correlated to the specified full mix strength by laboratory tests.

1.3.7 *Admixtures*

1.3.7.1 Pozzolans. It was earlier stated that prior to 1930, the use of pozzolanic material (pumicite) was given a trial in Big Dalton Dam by the Los Angeles Flood Control District. For Bonneville Dam completed by the Corps of Engineers in 1938, a portland-pozzolan cement was employed for all of the work. It was produced by intergrinding the cement clinker with a pozzolan processed by calcining an altered volcanic material at a temperature of about 1500 F (815 C). The proportion of clinker to pozzolan was 3:1 by weight. This type of cement was selected for use at Bonneville on the basis of results of tests on concrete which indicated large extensibility and low temperature rise. This is the only completed concrete dam in the United States known to the committee in which an interground portland-pozzolan cement has been employed.

In the intervening years, however, the use of pozzolan as a separate cementing material to be added at the mixer, in the ratio of one part pozzolan to two or three parts of Type II cement has come to be regular practice by the Bureau of Reclamation and more recently by the Corps of Engineers. Examples of dams in which pozzolans have been employed are given in Table 1.3.7.1.

TABLE 1.3.7.1 — EXAMPLES OF POZZOLAN USAGE

Name of dam	Type of pozzolan
Davis	Finely ground calcined opaline shale
Friant	Pumicite (without processing either by calcining or grinding)
Hungry Horse	Low carbon fly ash
Hartwell	Low carbon fly ash
Yellowtail	Low carbon fly ash
Glen Canyon	Finely ground pumice
John Day	Finely ground pumicite

Some experiments conducted by the Corps of Engineers indicate that for interior mass concrete, where stresses are moderately low, a much higher proportion of pozzolan to cement may be used when there is an economic advantage in so doing and still obtain the desired strength at the later ages.

For example, the results of laboratory tests indicate that an air-entrained mass concrete, containing 94 lb per cu yd (56 kg/m³) of cement and fly ash of equivalent solid volume to 188 lb per cu yd (112 kg/m³) of cement has produced a very workable mix, for which the water content was less than 100 lb per cu yd (60 kg/m³). The 1 year compressive strength of wet-screened 6 x 12-in. cylinders was in the order of 3000 psi (210 kg/m²). For such a mix the mass temperature rise would be exceedingly small. For gravity dams of moderate height, where the materials would be precooled so that the concrete as it reaches the forms will be, say, 15 fahrenheit deg (8 C) below the mean annual or rock temperature, there seems to be the possibility that neither longitudinal nor transverse contraction joints would be required. That is, the maximum temperature of the interior of the mass as hydration took place might not be appreciably greater than the mean annual temperature.

In one other respect the use of pozzolans has been shown to be advantageous—the substantial reduction in expansion of concretes containing reactive aggregates and high-alkali cements. The amount of this reduction has been found to vary with the character and fineness of the pozzolan and the amount employed; for some pozzolans the reduction may exceed 90 percent.

If reactive aggregates are employed, it is considered good practice to use both a low-alkali cement and a pozzolan of proven corrective ability.

1.3.7.2 Air-entraining agents. It became standard practice about 1945 to use purposely entrained air for concrete in most structures that were exposed to severe weathering conditions. This practice was applied to the concrete of exposed surfaces of dams as well as to the concrete pavements and reinforced concrete in general. However, because of the very favorable effect of entrained air on the workability of lean concrete mixes and also in reducing the water requirement and bleeding of fresh concrete, approved air-entraining agents introduced at the mixer have been employed for both interior and exterior concrete in the construction of practically all dams built in the United States during the past two decades.

Purposely entrained air has been the answer to the problem of obtaining satisfactory workability of mass concrete mixes of very low cement content. Most specifications for mass concrete now require that the quantity of entrained air, as determined on concrete samples wet-screened through the 1½-in. screen, shall be in the order of 5 percent.

1.3.7.3 Water-reducing, set-controlling admixtures. Within the last 10 years there has been a substantial increase in the use of water-reducing, set-controlling admixtures in mass concrete. Many such commercially available admixtures, either as derivatives of lignosulfonic acid or of hydroxylated carboxylic acid, have been found to impart physical properties to concrete which make their use beneficial. Chemical admixtures for concrete are covered by ASTM C 494.[2]

Benefits from the use of water-reducing, set-controlling admixtures used to date are principally in the areas of reduced water for a given slump and extension in the time of setting which lessens the likelihood of cold joints. In many instances the reduction in water permits a reduction in cement content that would otherwise be required to produce concrete of a required strength. This in turn lowers the total heat of hydration developed by the cement and hence reduces the temperature rise in the mass concrete.

Illustrative of major governmental agencies using water-reducing, set-controlling admixtures are the Bureau of Reclamation, the California Department of Water Resources, the Corps of Engineers, the Tennessee Valley Authority, the Air Force, and the Navy Bureau of Yards and Docks. In some instances their use is optional with the contractor; in others their use is mandatory by specification.

Projects of recent years in which such admixtures have been used include Glen Canyon Dam in Arizona, Morrow Point Dam in Colorado, Oroville Dam in California, Nickajack Dam in Tennessee, Guri Dam in Venezuela, Bhumiphol Dam in Thailand, and Peribonka No. 1 Dam in Canada, as well as many others in various parts of the world.

CHAPTER 2 — MATERIALS AND MIX PROPORTIONING

2.1 — Scope

2.1.1—As in the case with regular concrete, mass concrete is composed of cement, aggregate, and water and in some cases, pozzolans and other admixtures. The objective of mass concrete mixture proportioning is the selection of the types and quantities of these materials that will give economy and low temperature-rise potential with adequate workability for placing and adequate strength, durability, and impermeability to serve the intended purpose of the structure in which it is used. This chapter will describe materials that have been successfully used in mass concrete construction and factors influencing their selection and proportioning.

2.2 — Cements

2.2.1—The following types of hydraulic cement as covered by ASTM[3-5] and federal[6-10] specifications have been used in mass concrete construction.

(a) Portland cement: Types I, II, and IV

(b) Blended cement: Types IS and IP

(c) Cements other than portland cements: Slag cement and natural cement (used only with portland cement)

Also, mixtures of portland cement and pozzolan, portland cement and slag cement, and portland cement and natural cement batched separately on the job have been used in mass concrete construction. Economy and low temperature rise are both achieved by limiting the cement content to as small a value as possible.

2.2.2—Type I portland cement, also referred to as "normal," or "regular," or "standard" cement is the common type of cement usually used in ordinary construction.

2.2.3—Type II portland cement was developed for dam construction where moderate heat of hydration is desired. Specifications require that it contain no more than 8 percent tricalcium aluminate (C_3A), the compound that contributes substantially to early heat development in the concrete. Also, at the option of the purchaser the sum of tricalcium silicate (C_3S) and tricalcium aluminate may be limited to 58 percent or less and the heat of hydration to 70 calories per gram at 7 days and 80 calories per gram at 28 days.

2.2.4—Type IV portland cement, also referred to as "low heat" cement, is used mainly where it is desired to produce low heat development in massive structures. It has not been used in recent years because it has been found that in most cases heat development can be controlled satisfactorily by other measures. Type IV specifications limit the C_3A to 7 percent, the C_3S to 35 percent and place a minimum on the C_2S of 40 percent. The heat of hydration is limited to 60 calories per gram at 7 days and 70 calories per gram at 28 days.

2.2.5—Type IS portland blast-furnace slag cement is an intimate and uniform blend of portland cement and fine blast-furnace slag produced either by intergrinding portland cement clinker and granulated blast-furnace slag or by blending portland cement and finely ground blast-furnace slag. The amount of slag used may vary between 25 and 65 percent by weight of the portland blast-furnace slag cement. At the option of the purchaser the C_3A may be limited to 8 percent and the heat of hydration limited as in the case of Type II cement.

2.2.6—Type IP portland-pozzolan cement is an intimate and uniform blend of portland cement or portland blast-furnace slag cement and fine pozzolan produced either by intergrinding portland cement clinker and pozzolan or by blending portland cement or portland blast-furnace slag cement and finely divided pozzolan in which the pozzolan constituent is between 15 and 40 percent by weight of the portland-pozzolan cement.

2.2.7—Slag cement is finely divided material consisting essentially of an intimate and uniform blend of granulated blast-furnace slag and hydrated lime. The amount of granulated blast-furnace slag makes up at least 60 percent by weight of the slag cement. Because of its low strength producing characteristics, it is recommended that slag cement be blended with portland cement for making concrete.

2.2.8—Natural cement is the product obtained by finely pulverizing calcined argillaceous limestone. The temperature of calcination is no higher than is required to drive off carbonic acid gas. Natural cement is subject to lack of uniformity of its properties and is not recommended for use where uniform control is required. Natural cement is usually blended with portland cement for making concrete.

2.2.9—Low-alkali cements are defined by ASTM C 150 as portland cements containing not more than 0.60 percent alkalies calculated as the percentage of Na_2O plus 0.658 times the percentage of K_2O. These cements should be specified when the cement is to be used in concrete with aggregate that may be deleteriously reactive. Some engineers believe, that for more assured protection from reactive aggregate, the alkalies should be limited to 0.40 percent.

2.3 — Pozzolans

2.3.1—A pozzolan is defined as "A siliceous or siliceous and aluminous material which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitous properties." Pozzolans react chemically with the alkalies (K_2O and Na_2O) of the portland cement as well as with the calcium hydroxide or hydrated lime liberated during the hydration of portland cement to form a stable strength-producing cementitious compound. For best activity the siliceous ingredient of a pozzolan must be in an amorphous state such as glass or opal. Crystalline siliceous materials, such as quartz, do not combine readily with lime at normal temperature unless they are ground into a very fine powder. Pozzolans are covered by ASTM[11] and federal[12] specifications.

2.3.2 *Natural pozzolans*—Natural pozzolanic materials occur in large deposits throughout the western United States in the form of obsidian, pumicite, volcanic ashes, tuffs, clays, shales, and diatomaceous earth. These natural pozzolans usually require grinding; however, some of the volcanic materials are of suitable fineness in their natural state. The clays and shales, in addition to grinding, must be activated by calcining at temperatures in the range of 1200 to 1800 F (650 to 980 C).

2.3.3 *Fly ash*—Fly ash is the flue dust from powerplants burning ground or powdered coal. Suitable fly ash can be an excellent pozzolan; it has a low carbon content, a fineness about the same as that of portland cement, and occurs in the form of very fine, glassy spheres. Because of its shape and texture, the water requirement is usually reduced when fly ash is used in concrete. There are indications that in many cases the pozzolanic activity of the fly ash can be increased by cracking the glass spheres by means of grinding. However, this may also reduce its lubricating qualities and increase its water requirement.

2.3.4 *Use*—Pozzolans may be used to improve the workability and quality of concrete, to effect economy, or to protect against disruptive expansion caused by the reaction between certain aggregates and the alkalies in cement. Caution must be exercised in the selection and use of pozzolans, as their properties vary widely and some may introduce adverse properties into the concrete, such as excessive drying shrinkage and reduced early strength and durability. Before accepting a pozzolan for a specific job, it is advisable to test it in combination with the cement and aggregate to be used, so as to determine accurately the advantages or disadvantages of the pozzolan with respect to quality and economy of the concrete.

2.4 — Chemical admixtures

2.4.1—Admixtures are generally used to modify the properties of concrete so that it will be more suitable for a particular purpose. This modification may alter the characteristics of the fresh concrete or alter its properties after the concrete has hardened. Among the effects often sought with admixtures are: increased workability or reduced W/C, acceleration or retardation of setting time, acceleration of strength development, improved resistance to weather and chemical attack. A complete discussion of admixtures is given in the report of ACI Committee 212.[13] The chemical admixtures used in mass concrete may be grouped into three categories: accelerators, air-entraining agents, and water-reducing and set-controlling agents.

2.4.2 *Accelerators*—Accelerators are seldom used in mass concrete because generally early strength is not required and they contribute to early undesirable heat development in the mass. In rare cases, however, up to 1 percent calcium chloride by weight of the cement has been used to accelerate strength development in mass concrete during winter placing conditions.

2.4.3 *Air-entraining agents*—Most air-entraining admixtures are inexpensive soaps that develop bubbles in the concrete. Vinsol resin, a soap of wood resin, is the main ingredient of many brand name air-entraining admixtures. Sulfonated hydrocarbons, detergents, and salts of petroleum acids form the basis for some other air-entraining agents.

The entrainment of air greatly improves the workability of concrete, permits the use of harsher and more poorly graded aggregates and also those of undesirable shapes; i.e., flat, elongated. It reduces bleeding, and in general facilitates the placing and handling of mass concrete. Air-entrained concrete can be transported and placed with less segregation than regular concrete. Each percent of entrained air permits a reduction in mixing water of from 2 to 4 percent, with some improvement in workability and with no loss in slump. Entrained air substantially improves the resistance of concrete to deterioration caused by freezing and thawing and makes it somewhat more resistant to the passage of moisture than regular concrete.

Entrained air reduces the strength of most concrete. At the same water-cement ratio, strength is reduced by about 20 percent for the amounts of air recommended in ACI 613-54.[14] Where the cement content is held constant and advantage is taken of the reduced water requirement, the reduction in strength becomes negligible in lean mass concrete. Among the factors that influence the amount of air entrained in concrete for a given amount of agent are: grading and particle shape of the aggregate, richness of the mix, mixing time, slump and temperature of the concrete. For a given quantity of air-entraining admixture, air content increases with increase in slump up to 6 in. and decreases with increases in amount of fines, temperature of concrete, and mixing time.

2.4.4 *Chemical water-reducing and set-controlling agents*—Admixtures of the water-reducing type are materials generally consisting of certain organic compounds or mixtures which when added to portland cement concrete, markedly increase the fluidity of the concrete. These admixtures may be classified in the following categories:

1. Those using lignosulfonic acids and their salts as a base
2. Those using modifications and derivatives of lignosulfonic acids and their salts as a base
3. Those using hydroxylated carboxylic acids and their salts as a base
4. Those using modifications and derivatives of hydroxylated carboxylic acids and their salts as a base

The majority of water-reducing and set-controlling chemical admixtures fall into one of these categories. The unmodified acids and salts will generally reduce the water required to produce a given slump from 5 to 10 percent and will retard the initial set by at least 1 hr when used in amounts recommended by their manufacturers. Although the time of set is retarded, the strength of the concrete after 12 hr is usually equal to that of concrete containing no admixture. Depending on the richness of the mix, composition of the cement, the temperature and other factors, the use of a water-reducing agent will usually result in an appreciable increase in 1-, 7-, 28-day, and later-age strengths. This gain in strength cannot be explained by reductions in water-cement ratios alone; the chemicals apparently have some favorable effect on the hydration of the cement.

Water-reducing, set-controlling admixtures are used to keep the concrete plastic in massive blocks so that succesive layers can be placed and vibrated before the underlayer sets. They are also used to increase strength of concrete or produce the same strength with less cement.

2.5 — Aggregates

2.5.1 *Fine aggregate (sand)*—Fine aggregate is defined as aggregate passing the No. 4 (4.76 mm) sieve. It may be composed of natural grains, manufactured grains obtained by crushing larger size rock particles, or a mixture of the two. Fine aggregate should consist of hard, dense, durable, uncoated rock fragments. Fine aggregate should not contain harmful amounts of clay, silt, dust, mica, organic matter, or other impurities to such an extent that, either together or separately, they render it impossible to attain the required properties of concrete when employing normal proportions of the ingredients. For dams and other hydraulic structures the maximum percentage of the deleterious substances should be lower for face concrete in the zone of fluctuating water levels and for concrete in thin arch dams. It can be higher for concrete constantly immersed in water and for concrete in the interior of massive dams as well as for face concrete above the zone of fluctuating water levels.

Deleterious substances are usually limited to the following values:

	Percent by weight
Material passing No. 200 sieve	3
Lightweight material	2
Clay lumps	1
Total of other deleterious substances (such as alkali, mica, coated grains, soft flaky particles, and loam)	2

Aggregate grading has a definite effect on the workability of concrete. A good grading of sand for mass concrete will be within the limits shown in Table 2.5.1; however, laboratory investigation may show other gradings to be satisfactory. This permits a rather wide latitude in gradings for fine aggregate. Although the grading requirements themselves may be rather flexible, it is important that once the proportion is established, the grading of the sand be maintained reasonably uniform to avoid variations in the workability of the concrete.

TABLE 2.5.1 — GRADING LIMITS FOR FINE AGGREGATE

Sieve designation	Percentage retained, individual, by weight	Percentage retained, cumulative, by weight
⅜ in. (9.53 mm)	0	0
No. 4 (4.76 mm)	0 - 8	0 - 8
No. 8 (2.38 mm)	5 - 20	10 - 25
No. 16 (1.19 mm)	10 - 25	30 - 50
No. 30 (0.60 mm)	10 - 30	50 - 65
No. 50 (0.30 mm)	15 - 30	70 - 83
No. 100 (0.15 mm)	12 - 20	90 - 97
Pan fraction	3 - 10	100

2.5.2 *Coarse aggregate*—Coarse aggregate is defined as gravel, crushed gravel, or crushed rock, or a mixture of these, generally within the range of No. 4 (4.76 mm) to 6 in. (15 cm) in size. In general the maximum size of coarse aggregate should not exceed one-fourth of the least dimension of the structure concreted nor two-thirds of the least clear distance between reinforcement bars.

2.5.2.1—Coarse aggregate should consist of hard, dense, durable, uncoated rock fragments. Rock which is very friable or which tends to degrade during processing, transporting, or in storage should be avoided. Also, rock having an absorption greater than 3 percent or a specific gravity less than 2.5 are not considered suitable for mass concrete. Sulfates and sulfides, determined by chemical analysis and calculated as SO_3, should not exceed 0.5 percent by weight of the coarse aggregate. The percentages of other deleterious substances such as clay, silt, and fine dust in the coarse aggregate as delivered to the mixer should in general not exceed the following values:

	Percent by weight
Material passing No. 200 sieve	½
Lightweight material	2
Clay lumps	½
Other deleterious substances	1

2.5.2.2—Theoretically, the larger the maximum aggregate size the less cement is required in a given volume of concrete to achieve the desired quality. This theory is based on the fact that with well-graded materials the void space between the particles decreases as the range in sizes increases. However, it has been demonstrated (Fig. 2.5.2) that to achieve the greatest cement efficiency there is an optimum maximum size for each compressive strength level to be obtained with a given aggregate and cement.[15] While the maximum size of coarse aggregate for most structures is limited by the configuration of the forms and reinforcing steel, in most mass concrete structures these requirements permit

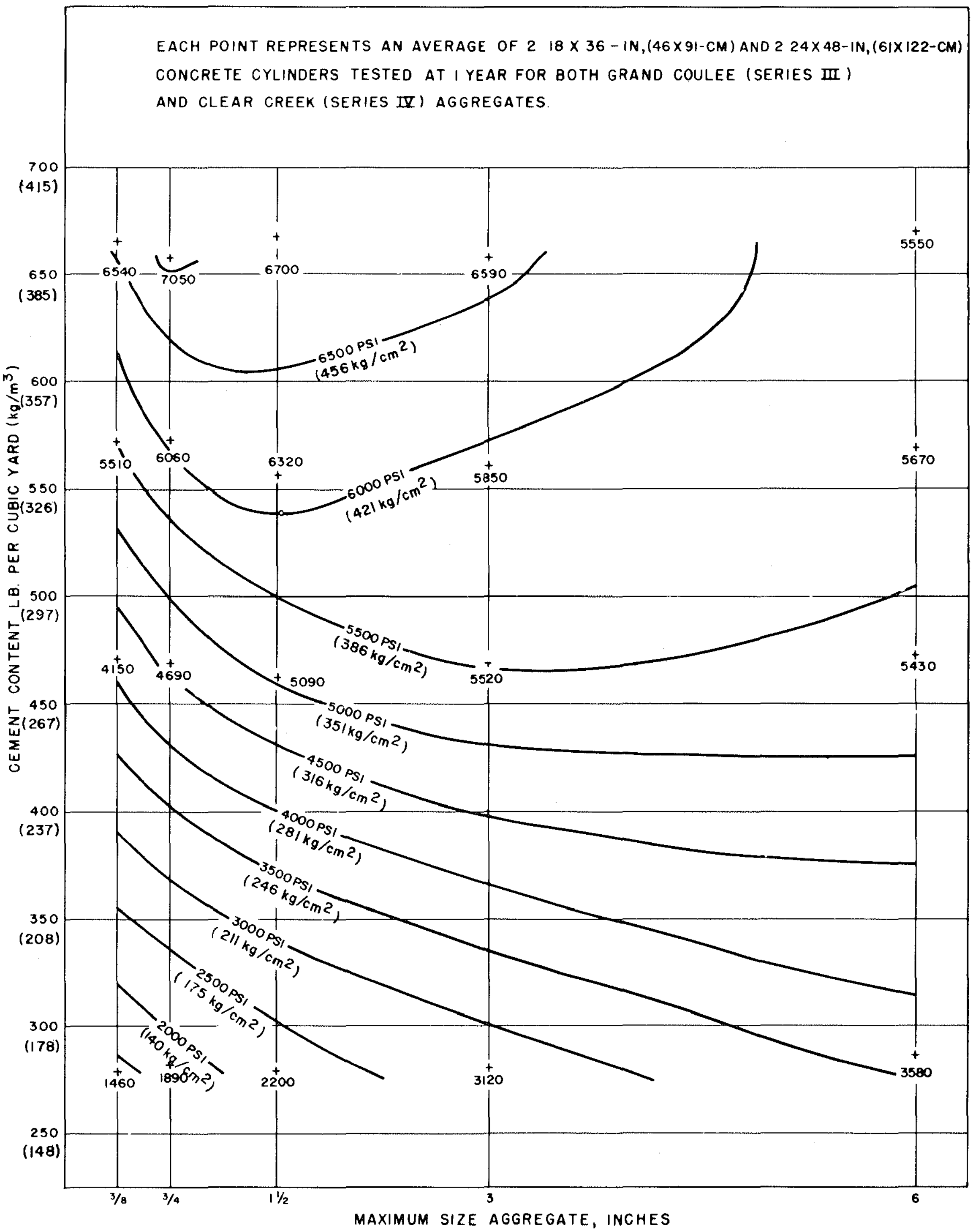

Fig. 2.5.2 — Effect of aggregate size and cement content on compressive strength at 1 year (from Higginson, *et al.*, Reference 15)

an almost unlimited maximum aggregate size. In addition to availability, the economical maximum size is therefore determined by the design strength and plant requirements for processing, batching, mixing, transporting, placing, and consolidating the concrete. Large aggregate particles of irregular shape tend to promote cracking around the larger particles because of differential volume change. They also cause voids to form underneath them due to water and air accumulating during vibration. Although larger sizes have been used on occasion, an aggregate size of 6 in. (15 cm) has been normally adopted as the maximum practical size.

2.5.2.3 The particle shape has an important bearing on workability and consequently, on water requirement. Rounded particles, such as those which occur in deposits of streamworn sand and gravel, provide best workability. However, modern crushing and grinding equipment is capable of producing both fine and coarse aggregate of entirely adequate particle shape from ledge rock. Thus, in spite of the slightly lower water requirement of natural rounded aggregates, it is seldom economical to import natural aggregates when a source of high quality crushed aggregate is available near the site of the work. It is necessary, however, to determine that an adequate crushing job is being obtained. One procedure commonly used is to include in coarse aggregate specifications a requirement limiting flat and elongated particles to 25 percent in each size group. A flat particle is defined as one having a ratio of width to thickness greater than three, while an elongated particle is defined as one having a ratio of length to width greater than three.

2.5.2.4 Aggregate grading has a definite effect on the workability of concrete, and it is one of the factors over which the concrete technician has some control. In United States practice it is customary to divide coarse aggregate into the following sizes:

Cobbles	6 in. to 3 in.	(152 mm to 76 mm)
Coarse fraction	3 in. to 1½ in.	(76 mm to 38 mm)
Medium fraction	1½ in. to ¾ in.	(38 mm to 19 mm)
Fine fraction	¾ in. to No. 4	(19 mm to 4.76 mm)

Sizes are satisfactorily graded when one-third to one-half of the aggregate within the limiting screens is retained on an intermediate screen.

Distribution of the particles among the sizes may be accomplished by trial or by any one of several theories. A commonly used theoretical approach is the exponential grading characterized by the following equation:

$$P = \left(\frac{d}{D}\right)^n \times 100$$

where

d = sieve opening
D = maximum size of aggregate
P = cumulative percent passing sieve size d
n = factor related to the particle shape and texture of the coarse aggregate

The exponent n is usually in the range of 0.4 to 0.5 with rounded aggregate assigned the higher number. The theoretical methods are only devices for obtaining an initial approximate coarse aggregate grading. Experience has shown that a rather wide range of coarse aggregate gradings may be used (Table 2.5.2). Workability is frequently improved by reducing the proportion of cobbles called for by the theoretical gradings. When natural gravel is used, it is economically desirable to depart from theoretical gradings to approximate as closely as workability permits the average grading of material in the deposit. Where there are extreme excesses or deficiencies in a particular size, it is preferable to waste a portion of the material rather than to produce unworkable concrete. The problem of waste usually does not occur when the aggregate is crushed stone. With modern two- and three-stage crushing it is normally possible to adjust the operation so that a workable grading is obtained. Unless finish screening is employed, it is well to reduce the amount of the finest size of coarse aggregate since that size contains the accumulated undersize of the larger sizes. However, finish screening is strongly recommended for aggregate for mass concrete. With finish screening, there is little difficulty meeting specifications which limit undersize to 4 percent in the cobble size, 3 percent in the intermediate sizes, and 2 percent in the fine size.

2.5.2.5 In some parts of the world "gap" gradings are used in mass concrete. These are gradings in which the material in one or more sieve sizes is missing. In United States practice continuous gradings are normally used.

TABLE 2.5.2 — GRADING LIMITS FOR COARSE AGGREGATES IN MASS CONCRETE

	Percentage of coarse aggregate fractions (clean separation)					
				Fine		
Maximum size aggregate in concrete, in. (mm)	Cobbles 3-6 in. (76-152 mm)	Coarse 1½ to 3 in. (38-76 mm)	Medium ¾ to 1½ in. (19-38 mm)	3/16 in. (#4) to ¾ in. (5-19 mm)	⅜ to ¾ in.* (10-19 mm)	3/16 in. (#4) to ⅜ in.* (5-10 mm)
¾ (19 mm)	0	0	0	100	55-73	27-45
1½ (38 mm)	0	0	40-55	45-60	30-35	15-25
3 (76 mm)	0	20-40	20-40	25-40	15-25	10-15
6 (152 mm)	20-35	20-32	20-30	20-35	12-20	8-15

*These columns used only when fine gravel is separated into two sizes.

Gap gradings can be used economically where the material occurs naturally gap-graded. But comparisons which can be made between concretes containing gap-graded aggregate and continuously graded aggregate indicate there is no advantage in purposely producing gapgradings. Continuous gradings produce more workable mass concrete requiring somewhat lower slump, less water and cement and having higher compressive strength. Continuous gradings can always be produced from crushing operations, and most natural aggregate deposits in the United States contain material from which continuous gradings can be economically prepared.

2.6 — Water

2.6.1—Water used for mixing concrete should be free of materials that significantly affect the hydration reactions of portland cement or that otherwise interfere with the phenomena that are intended to occur during the mixing, placing, and curing of concrete. Water that is fit to drink may generally be regarded as acceptable for use in mixing concrete. When it is desired to determine whether a water contains materials that significantly affect the strength development of cement, tests should be made comparing the compressive strength of mortars made with water from the proposed source with that of mortars made with distilled water. If the average of the results of these tests on specimens containing the water being evaluated is less than 90 percent of that obtained with specimens containing distilled water, the water represented by the sample should not be used for mixing concrete. If a potential water source lacking a service record is so unusual as to contain amounts of impurities as large as 5000 to 10,000 ppm, or more, then, to insure durable concrete, tests for volume stability (length change) may be advisable as well as for strength.[16]

Waters containing up to several thousand parts per million of normally found mineral acids such as hydrochloric acid or sulfuric acid can be tolerated so far as strength development is concerned.[16] Waters containing even very small amounts of various sugars or sugar derivatives should not be used as set may be retarded unacceptably. The harmfulness of such waters may be revealed in the comparative strength tests.

2.7 — Selection of proportions

2.7.1—The procedure generally used for establishing a mix for mass concrete is to select mix proportions by the trial mix method, generally following the recommendations of ACI Committee 211.[14]

2.7.2 *Water-cement ratio*—The key to proportioning is the selection of the water-cement ratio which will provide adequate strength, durability, and impermeability. In addition, there must be sufficient fine material to provide proper placeability. Experience has shown that with the best shaped aggregates of 6 in. (15 cm) maximum size the quantity of cement-size material required for workability is about 210 lb per cu yd (125 kg/m^3) of concrete while angular aggregates require at least 235 lb per cu yd (140 kg/m^3). Although a lower cement factor may be calculated from the required water-cement ratio and the observed water requirement for the job materials, these figures should be used until field trials demonstrate that the cement factor may be safely reduced.

2.7.3 *Trial batch weights*—The first step in arriving at the actual batch weights is to select the maximum aggregate size for each part of the work. Criteria for this selection are given in Section 2.5.2. The next step is to assume or determine the total water content required to provide a slump of from 1½ in. (3.8 cm) to 2 in. (5 cm). [In tests for slump, aggregate larger than 1½ in. must be removed by screening the wet concrete. Slumps in excess of 2 in. (5 cm) and 2½ in. (6 cm) for 6 in. (15 cm) and 3 in. (7.5 cm) maximum size aggregates respectively are susceptible to undesirable segregation in handling.] For 6 in. maximum size aggregate, water contents for air-entrained concrete may vary from about 120 to 150 lb per cu yd (71 to 89 kg/m^3) for natural aggregates, and from 140 to 180 lb per cu yd (83 to 107 kg/m^3) for crushed aggregates. Corresponding water requirements for 3 in. maximum size aggregate are approximately 20 percent higher.

The batch weight of cement is the quotient of the total weight of water divided by the water-cement ratio or, when workability governs, is merely the minimum weight of cement required to satisfactorily place the concrete (see Section 2.7.2). With the batch weights of cement and water determined and with an assumed air content of 3 to 5 percent, the remainder of the material is aggregate. The only remaining decision is to select the relative proportions of fine and coarse aggregate. The optimum proportions depend on aggregate grading and particle shape, and they can be finally determined only in the field. For 6 in. aggregate concrete containing natural sand and gravel, the ratio of fine aggregate to total aggregate by absolute volume may be as low as 21 percent. With crushed aggregates the ratio may be in the range 25 to 27 percent.

2.7.4 *Pozzolans*—Mixture proportioning procedures do not change appreciably when a pozzolan is included as a part of the cementitious materials. Attention must be given to the following matters:

(a) water requirement may change; (b) very early strength may become critical; and (c) for maximum economy the age at which design strength is attained should be greater. Concrete containing pozzolan gains strength somewhat slower than concrete made with straight portland cement; however, the load on mass concrete is generally not applied until the concrete is relatively old. Therefore, mass concrete containing pozzolan is usually designed on the basis of 90-day to 1-year strengths. While mass concrete does not require strength at early ages to perform its design function, most systems of construction require that the forms for each lift be anchored to the next lower lift. Therefore, the early strength must be great enough to prevent pullout of the form anchors. Specially designed form anchors may be required if the 3-day strength is less than 350 psi (25 kg/cm^2), which may occur when large amounts of pozzolan are used.

2.8 — Temperature control

The principal technical problem peculiar to mass concrete is the probability of high tensile stresses resulting from heat generated by the hydration of cement with subsequent differential cooling. Control of temperature drop is achieved by controlling placing temperature, limiting the temperature-rise potential of the concrete, controlling lift thickness and placing schedule, and removal of heat through embedded cooling coils. It is practical to cool coarse aggregate, somewhat more difficult to cool fine aggregate, and practical to batch a portion or all of the added mixing water in the form of ice. As a result, placing temperatures of as low as 50 F (10 C) are quite easily obtained and commonly specified. Lower temperatures are obtainable with more difficulty. Cooled concrete is advantageous in mixture proportioning since water requirement usually decreases as temperature drops.

The chief means for limiting temperature rise is controlling the type and amount of cementitious materials. The ASTM specification for Type II portland cement contains an option which makes it possible to limit the heat of hydration to 70 calories per gram at 7 days and 80 calories per gram at 28 days. Use of a pozzolan as a replacement further delays and reduces heat generation. This delay is an advantage except that when cooling coils are used the period of operation must be extended. If the mixture can be proportioned so that the cementitious materials can be limited to not more than 235 lb per cu yd (140 kg/m^3), the temperature rise for most concretes will not exceed 35 F (19 C). A complete discussion of temperature control is given in Chapter 5.

CHAPTER 3 — PROPERTIES

3.1—General

3.1.1—The design and construction of massive concrete structures, especially dams, is influenced by the topography and foundation characteristics of the site and availability of suitable materials of construction. Economics, subservient only to safety requirements, is the most important single parameter to consider. Economics may dictate the choice of type of structure for a given site. The proportioning of the concrete mix is, in turn, governed by the requirements of the type of structure and such properties as the strength, durability, and thermal properties. For large structures extensive investigations of aggregate, admixtures, and pozzolan are justified. Concrete mix investigations to determine the most economical proportions of selected ingredients to produce the desired properties of the concrete are necessary.

3.1.2—The specific properties of concrete which should be known are compressive strength, modulus of elasticity, Poisson's ratio, triaxial shear strength, volume change during drying, thermal coefficient of expansion, specific heat, thermal conductivity, diffusivity, permeability, and durability. Approximate values of these properties based on computations or past experience are often used in preliminary evaluations. Useful as such approximations may be, the complex heterogeneous nature of concrete and the physical and chemical interactions of aggregate and paste are still not sufficiently known to permit computation of reliable values. For this reason, it is again emphasized that extensive laboratory and field investigations must be conducted to assure a safe structure at lowest cost.

3.1.3—Search of the literature[17-32] for properties of mass concrete produced only a limited amount of data, other than strength, except for some dams in the United States. A compilation of mix proportion data on representative dams is given in Table 3.1.3.[17,20,25,27] Reference will be made to concrete mixes described in Table 3.1.3 in discussions of properties reported in Tables 3.2.1 through 3.8.1.

3.2 — Compressive strength

3.2.1—The water-cement ratio to a large extent governs the quality of the hardened portland cement binder. Strength, impermeability, and most other sought-for properties of concrete are improved by lowering the water-cement ratio. A comparison of strength data given in Table 3.2.1 shows a considerable variation from a uniform relationship between water-cement ratio and strength. Factors, totally or partially independent of the water-cement ratio, which affect the

TABLE 3.1.3 — CONCRETE MIXES OF 23 DAMS AND RELATED INFORMATION

No.	Name	Year completed	Type	Cement Type	Cement lb/cu yd, (kg/m³)	Pozzolan Type	Pozzolan lb/cu yd, (kg/m³)	Sand lb/cu yd, (kg/m³)	Coarse aggregate lb/cu yd, (kg/m³)	Coarse aggregate Type	MSA, in., (cm)	Water lb/cu yd, (kg/m³)	W/C	Entrained air, percent	Parts aggregate	Density lb/cu yd, (kg/m³)	WRA admixture used
1	Hoover	1936	Arch gravity	IV	380 (225)	—	0	931 (552)	2679 (1589)	Limestone and granite	9.0 (22.9)	220 (130)	0.58	0	9.5	4210 (2497)	No
2	Grand Coulee	1942	Straight gravity	II and IV	377 (224)	—	0	982 (582)	2568 (1523)	Basalt	6.0 (15.2)	226 (134)	0.60	0	9.4	4153 (2463)	No
3	Friant	1942	Straight gravity	IV	300 (178)	Pumicite	60 (36)	942 (559)	2634 (1562)	Quartzite, granite, and rhyolite	8.0 (20.3)	214 (127)	0.59	0	9.9	4150 (2461)	No
4	Shasta	1945	Curved gravity	IV	370 (219)	—	0	906 (537)	2721 (1614)	Andesite and slate	6.0 (15.2)	206 (122)	0.56	0	9.8	4203 (2492)	No
5	Hungry Horse	1952	Arch gravity	II	188 (111)	Fly Ash	90 (53)	842 (499)	2820 (1672)	Sandstone	6.0 (15.2)	130 (77)	0.47	3.0	13.2	4070 (2414)	No
6	Glen Canyon	1963	Arch gravity	II	188 (111)	Pumicite	94 (56)	777 (461)	2784 (1651)	Limestone, chalcedonic chert, and sandstone	6.0 (15.2)	153 (91)	0.54	3.5	12.6	3996 (2370)	No
6A	(0.37 percent admixture added)	1963	Arch gravity	II	188 (111)	Pumicite	90 (53)	800 (474)	2802 (1662)		6.0 (15.2)	140 (83)	0.50	3.5	13.0	4020 (2384)	Yes
7	Flaming Gorge	1962	Arch gravity	II	188 (111)	Calc. shale	94 (56)	729 (432)	2900 (1720)	Limestone and sandstone	6.0 (15.2)	149 (88)	0.53	3.5	12.9	4060 (2408)	No
8	Yellowtail	1965	Arch gravity	II	197 (117)	Fly Ash	85 (50)	890 (528)	2817 (1670)	Limestone and andesite	6.0 (15.2)	139 (82)	0.49	3.0	13.1	4128 (2448)	No
9	Morrow Point	1967	Thin Arch	II	373 (221)	—	0	634 (376)	2851 (1691)	Andesite, tuff, and basalt	4.5 (11.4)	156 (93)	0.42	4.3	9.3	4015 (2381)	Yes
10	Bartlett	1939	Multiple Arch	IV	466 (276)	—	0	1202 (713)	2269 (1346)	Quartzite, and granite	3.0 (7.6)	270 (160)	0.58	0	7.5	4180 (2479)	No
11	Bonneville	1938	Gravity	Portland pozzolan	329 (195)	—	0	1094 (649)	2551 (1513)	Basalt	6.0 (15.2)	251 (149)	0.76	0	11.1	4223 (2504)	No
12	Detroit	1953	Straight gravity	II and IV	226 (134)	—	0	1000 (593)	2690 (1595)	Diorite	6.0 (15.2)	191 (113)	0.85	5.5	16.3	4107 (2435)	No
13	Norris	1936	Straight gravity	II	338 (200)	—	0	1264 (750)	2508 (1487)	Dolomite	6.0 (15.2)	227 (135)	0.67	0	11.2	4212 (2498)	No
14	Kentucky	1944	Straight gravity	II	338 (200)	—	0	967 (573)	2614 (1550)	Limestone	6.0 (15.2)	213 (126)	0.63	0	10.6	4136 (2453)	No
15	Hartwell	1961	Gravity	II	165 (98)	Fly Ash	57 (34)	797 (472)	2880 (1708)	Granite	6.0 (15.2)	183 (108)	0.82	5.5	16.6	4082 (2421)	No
16	John Day	U.C.	Gravity	II	148 (88)	Calc. shale	50 (30)	937 (554)	2970 (1761)	Nat. gravel	6.0 (15.2)	135 (80)	0.68	6.0	19.7	4240 (2515)	No
17	Salamonde (Portugal)	1953	Thin Arch	II	421 (250)	—	0	739 (438)	2621 (1554)	Granite	7.9 (20.0)	225 (133)	0.54	0	8.0	4006 (2376)	
18	Pieve di Cadore (Italy)	1949	Arch gravity	Ferric-pozzolanio	253 (150)	Natural	84* (50)	1180 (700)	2089 (1239)	Limestone	4.7 (12.0)	213 (126)	0.63	2.0	9.7	4316 (2560)	Yes
19	Rossens (Switzerland)	1948	Arch	I	421 (250)	—	0			Limestone	3.1 (8.0)	225 (133)	0.53	0			No
20	Chastang (France)	1951	Arch gravity	250/315	379 (225)			759 (450)	2765 (1640)	Granite	9.8 (25.0)	169 (100)	0.45		9.3	4072 (2415)	
21	Warragamba (Australia)	1960	Straight gravity	II	330 (196)	—	0	848 (503)	2845 (1687)	Prophyry and quartzite	6.0 (15.2)	175 (104)	0.53	0	11.2	4163 (2469)	No
22	Francisco Madero (Mexico)	1949	Round-Head Buttress	IV	372 (221)			893 (530)	2381 (1412)	Rhyolite and basalt	6.0 (15.2)	223 (132)	0.60		8.8		
23	Krasnoiarsk (USSR)	U.C.†	Straight gravity	IV and portland blast-furnace	388 (230)	—	0			Granite	3.9 (10.0)	213 (126)	0.55				Yes

*Pozzolan interground with cement (25 percent pozzolan for the summer months). †Data gathered in 1964.

TABLE 3.2.1 — CEMENT/WATER REQUIREMENTS AND STRENGTHS OF CONCRETES IN VARIOUS DAMS

Dam	Country	Cement lb/cu yd (kg/m³)	Water lb/cu yd (kg/m³)	Predominant aggregate type	MSA, in. (cm)	W/C	90-day str psi (kg/cm²)	Cement efficiency psi/lb (kg/cm²/kg)
La Palisse	France	506 (300)	250 (148)	Granite	4.7 (12.0)	0.49	4790 (337)	9.5 (1.1)
Le Gage	France	590 (350)	253 (150)	Granite	4.7 (12.0)	0.43	5060 (356)	8.6 (1.0)
Chastang	France	379 (225)	169 (100)	Granite	9.8 (25.0)	0.45	3770 (265)	10.0 (1.2)
Tignes	France	349 (207)	190 (113)	Granite	7.9 (20.0)	0.54	4250 (299)	12.2 (1.4)
L'Aigle	France	379 (225)	211 (125)	Granite	9.8 (25.0)	0.56	3200 (225)	8.5 (1.0)
Barrea	Italy	500 (297)	225 (133)	Limestone	3.2 (8.0)	0.45	5000 (352)	10.0 (1.2)
Pieve di Cadore	Italy	337 (200)	213 (126)	Dolomite	4.0 (10.0)	0.63	6400 (450)	19.0 (2.3)
Forte Baso	Italy	404 (240)	238 (141)	Porphyry	3.8 (9.6)	0.59	4920 (346)	12.2 (1.4)
Lumiei	Italy	455 (270)	226 (134)	Limestone	3.1 (8.0)	0.50	5670 (399)	12.5 (1.5)
Cabril	Portugal	370 (220)	195 (116)	Granite	5.9 (15.0)	0.53	4150 (292)	11.2 (1.3)
Bouca	Portugal	420 (249)	195 (116)	Granite	5.9 (15.0)	0.46	5500 (387)	13.1 (1.6)
Salamonde	Portugal	420 (249)	225 (133)	Granite	7.9 (20.0)	0.54	4250 (299)	10.1 (1.2)
Canicada	Portugal	420 (249)	225 (133)	Granite	7.9 (20.0)	0.54	4650 (327)	11.1 (1.3)
Castelo Bode	Portugal	370 (220)	180 (107)	Quartzite	7.9 (20.0)	0.49	3800 (267)	10.3 (1.2)
Rossens	Switzerland	420 (249)	225 (133)	Glacial mix	2.5 (6.4)	0.54	5990 (421)	14.3 (1.7)
Mauvoisin	Switzerland	319 (189)	162 (96)	Gneiss	3.8 (9.6)	0.51	4960 (349)	15.5 (1.8)
Zervreila	Switzerland	336 (199)	212 (126)	Gneiss	3.8 (9.6)	0.63	3850 (271)	10.5 (1.4)
Hungry Horse	USA	188-90 (111-53)	130 (77)	Sandstone	6 (15.2)	0.47	3100 (218)	11.1 (1.3)
Glen Canyon	USA	188-94 (111-56)	153 (91)	Limestone	6 (15.2)	0.54	3810 (268)	13.5 (1.6)
Flaming Gorge	USA	188-94 (111-56)	149 (88)	Limestone and sandstone	6 (15.2)	0.53	3500 (246)	12.4 (1.5)
Krasnoiarsk	USSR	338 (230)	213 (126)	Granite	3.9 (10.0)	0.55	3280 (230)	8.4 (1.0)

TABLE 3.3.2 — COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES OF MASS CONCRETE

	Compressive strength, psi (kg/cm²)				Elastic properties							
					Modulus of elasticity, E × 10^{-6} psi (E × 10^{-6} kg/cm²)				Poisson's ratio			
	Age, days				Age, days				Age, days			
	28	90	180	365	28	90	180	365	28	90	180	365
1. Hoover	3030 (213)	3300 (232)	—	4290 (302)	5.5 (0.39)	6.2 (0.44)	—	6.8 (0.48)	0.18	0.20	—	0.21
2. Grand Coulee	4780 (336)	5160 (363)	—	5990 (421)	4.7 (0.33)	6.1 (0.43)	—	6.0 (0.42)	0.17	0.20	—	0.23
3. Friant	4000 (281)	—	4240 (298)	4170 (293)	5.5 (0.39)	—	4.7 (0.33)	5.9 (0.41)	—	—	—	—
4. Shasta	4210 (296)	4650 (327)	—	5140 (361)	4.9 (0.34)	5.1 (0.36)	—	5.7 (0.40)	—	—	—	—
5. Hungry Horse	2660 (187)	3100 (218)	3800 (267)	3850 (271)	4.4 (0.31)	4.6 (0.32)	4.8 (0.34)	4.3 (0.30)	0.16	0.17	0.18	0.18
6. Glen Canyon	2550 (179)	3810 (268)	3950 (278)	—	5.4 (0.38)	—	5.8 (0.41)	—	0.11	—	0.14	—
6A. Glen Canyon*	3500 (246)	4900 (344)	6560 (461)	6820 (479)	5.3 (0.37)	6.3 (0.44)	6.7 (0.47)	—	0.15	0.15	0.19	—
7. Flaming Gorge	2950 (207)	3500 (246)	3870 (272)	4680 (329)	3.5 (0.25)	4.3 (0.30)	4.6 (0.32)	—	0.13	0.25	0.20	—
8. Yellowtail	—	4580 (322)	5420 (381)	5640 (396)	—	6.1 (0.43)	5.4 (0.38)	6.2 (0.44)	—	0.24	0.26	0.27
9. Morrow Point*	4770 (335)	5960 (419)	6430 (452)	6680 (470)	4.4 (0.31)	4.9 (0.35)	5.3 (0.38)	4.6 (0.33)	0.22	0.22	0.23	0.20
10. Bartlett	3200 (225)	—	7540 (530)	7620 (536)	—	—	4.1 (0.29)	4.4 (0.31)	—	—	—	—

*Water-reducing agent added.

TABLE 3.4.2 — ELASTIC PROPERTIES OF MASS CONCRETE

Age at time of loading	Instantaneous and sustained modulus of elasticity,* psi (kg/cm²) $\times 10^{-6}$														
	Grand Coulee			Shasta			Hungry Horse			Canyon Ferry			Monticello		
	E	E^1	E^2	E	E^1	E^2	E	E^1	E^2	E	E^1	E^2	E	E^1	E^2
2 days	1.7 (0.12)	0.83 (0.058)	0.76 (0.053)	1.4 (0.098)	0.54 (0.038)	0.49 (0.034)	2.8 (0.20)	1.5 (0.10)	1.4 (0.098)	2.8 (0.20)	1.3 (0.091)	1.2 (0.084)	1.4 (0.098)	0.63 (0.044)	0.58 (0.041)
7 days	2.3 (0.16)	1.1 (0.077)	1.0 (0.070)	2.1 (0.15)	1.0 (0.070)	0.96 (0.067)	4.2 (0.30)	1.9 (0.13)	1.8 (0.13)	3.8 (0.27)	1.8 (0.13)	1.6 (0.11)	2.2 (0.15)	0.94 (0.066)	0.85 (0.060)
28 days	3.5 (0.25)	1.8 (0.13)	1.6 (0.11)	3.5 (0.25)	1.8 (0.13)	1.6 (0.11)	4.5 (0.32)	2.6 (0.18)	2.4 (0.17)	4.6 (0.32)	2.5 (0.18)	2.4 (0.17)	3.6 (0.25)	1.8 (0.13)	1.7 (0.12)
90 days	4.1 (0.29)	2.5 (0.18)	2.3 (0.16)	4.4 (0.31)	2.7 (0.19)	2.5 (0.18)	5.2 (0.37)	3.2 (0.22)	3.0 (0.21)	5.6 (0.39)	3.4 (0.24)	3.2 (0.22)	4.2 (0.30)	2.6 (0.18)	2.4 (0.17)
1 year	5.0 (0.35)	3.1 (0.22)	2.9 (0.20)	4.7 (0.33)	3.5 (0.25)	3.4 (0.24)	5.7 (0.40)	3.6 (0.25)	3.4 (0.24)	6.7 (0.47)	3.9 (0.27)	3.6 (0.25)	4.6 (0.32)	3.1 (0.22)	2.9 (0.20)
5 years	5.3 (0.37)	3.6 (0.25)	3.4 (0.24)				5.9 (0.41)	4.0 (0.28)	3.8 (0.27)	7.2 (0.51)	4.6 (0.32)	4.3 (0.30)	4.8 (0.34)	3.3 (0.23)	3.2 (0.22)
7¼ years				5.6 (0.39)	4.3 (0.30)	4.1 (0.29)									

*All concretes mass mixed, wet screened to 1½ in. (3.8 cm) maximum size aggregate.
E = instantaneous modulus of elasticity at time of loading
E^1 = sustained modulus after 365 days under load
E^2 = sustained modulus after 1000 days under load

strength are: (1) type and brand of cement, (2) amount and type of pozzolan, (3) surface texture and shape of the aggregate, (4) the mineralogic makeup and strength of the aggregate, (5) aggregate grading, and (6) the improvement of strength by admixtures above that attributable to a reduction in water-cement ratio.

3.2.2—High concrete strengths are usually not required in mass concretes except for thin arch dams. Mix proportioning should determine the minimum cement content for adequate strength to give greatest economy and minimum temperature rise. Cement requirements for adequate workability and durability frequently govern the portland cement content rather than the strength.

3.2.3—Mass concrete is seldom required to withstand substantial stress at early age. Therefore, to take full advantage of the strength properties of the cementing materials the design strength is usually based on the strength at ages from 90 days to 1 year. Job-control cylinders must of necessity be tested at an earlier age if they are to be useful in exercising control and uniformity in the concrete being placed. For the sake of convenience job-control test specimens are usually 6 x 12-in. (15 x 30 cm) cylinders or 8 x 8-in. (20 x 20 cm) cubes containing concrete wet screened to 1½ in. (38 mm) maximum-size aggregate. Correlation tests should, therefore, be made well in advance of construction between the strength of wet screened concrete tested at 28 days and test specimens not smaller than 18 x 36 in. (46 x 91 cm) containing the full mass concrete tested at the design test age. The strength of large test specimens will usually be 80 to 90 percent of the strength of 6 x 12-in. cylinders tested at the same age. Accounting for the continued strength development beyond 28 days, particularly where pozzolans are employed, the correlation factors at 1 year may range from 1.15 to 3.0 times the strength of the wet screened control specimens tested at 28 days.

3.2.4—The factors involved in relating results of strength tests on small samples to the probable strength of mass concrete structures are several and complex and still essentially unresolved. Because of these complexities, concrete strength requirements are usually several times the calculated maximum design stresses for mass concrete structures.

3.3 — Elastic properties

3.3.1—Concrete is not a truly elastic material, and the graphic stress-strain relationship for continuously increasing load is generally in the form of a curved line. However, the modulus of elasticity is for practical purposes considered a constant within the range of stresses to which mass concrete is usually subjected.

3.3.2—The modulus of elasticity of concrete representative of various dams is given in Table 3.3.2. These values range from 3.5 to 5.5 $\times$ 10^6 psi (0.25 to 0.39 $\times$ 10^6 kg/cm^2) at 28 days and from 4.3 to 6.8 $\times$ 10^6 psi (0.30 to 0.48 $\times$ 10^6 kg/cm^2) at 1 year. Usually, concretes having higher strengths have higher values of elastic modulus and show a general correlation of increase in modulus with increase in strength, although modulus of elasticity is not directly proportional to strength. The modulus of elasticity of concrete is to some extent dependent on the modulus of elasticity of the aggregate. However, for a given cement paste the modulus of elasticity of the aggregate has less effect on the modulus of elasticity of the concrete than can be accounted for by the volumetric proportions of the aggregate.[33]

Modulus of elasticity for a given concrete exhibits a much higher coefficient of variation than the compressive strength. The greater variation results in part from the greater inaccuracies of the test procedures necessary to measure small strains on a heterogeneous mixture containing large size aggregate.

3.3.3—Poisson's ratio data given in Table 3.3.2 tend to range between the values of 0.16 to 0.20 with generally a small increase with increasing time of cure. Extreme values may vary from 0.11 to 0.27. Poisson's ratio, like modulus of elasticity, varies with the Poisson's ratio of the aggregate, the cement paste, and the relative proportions of the two.

3.3.4—The growth of internal microcracks in concrete under load commences at compressive stresses equal to about 35 to 50 percent of the nominal compressive strength under short term loading. Above this stress, the over-all volumetric strain reflects the volume taken up by these internal fissures, and Poisson's ratio and the elastic moduli are no longer constant.

3.4 — Creep

3.4.1—Creep of concrete is deformation that occurs while concrete is under sustained stress. Creep appears to be mainly related to the modulus of elasticity of the concrete. Concretes having high values of modulus of elasticity generally have low values of creep deformation and concretes having a low value of modulus of elasticity show greater amounts of creep deformation.

3.4.2—One method of expressing the effect of creep is as the sustained modulus of elasticity of the concrete in which the stress is divided by the total deformation for the time under load. The instantaneous and sustained modulus of elasticity values obtained on 6 in. diameter by 16-in. (15 x 40 cm) creep test cylinders made with wet screened concrete, 1.5 in. (38 mm) maximum-size aggregate, are recorded in Table 3.4.2. The instantaneous modulus is measured immediately after the concrete is subjected to load. The sustained modulus represents values after 365 and 1000 days under load. The sustained modulus is approximately one-half that of the instantaneous modulus when load is applied at early ages and is a slightly higher percentage of the instantaneous modulus when the loading age is 90 days and greater. Creep of concrete appears to be approximately directly proportional to the applied stress up to about 40 percent of the ultimate strength of the concrete.

3.5 — Volume change

3.5.1—Volume changes are caused by changes in moisture content of the concrete, chemical reactions, changes in temperature, and stresses from applied loads. Excessive volume change is detrimental to concrete. Cracks are formed in

TABLE 3.5.1 — VOLUME CHANGE AND PERMEABILITY OF MASS CONCRETE

Structure	Autogenous volume change		Drying shrinkage	Permeability K_q*
	90 days, millionths	1 year, millionths	1 year, millionths	
Hoover	—	—	—270	0.62×10^{-4}
Grand Coulee	—	—	—420	—
Angostura	+3	0	—390	—
Kortes	+14	—23	—600	—
Hungry Horse	—44	—52	—520	1.85×10^{-4}
Canyon Ferry	+6	—37	—397	1.93×10^{-4}
Monticello	—15	—38	—998	8.20×10^{-4}
Anchor	—33	—36	—588	45.2×10^{-4}
Glen Canyon	—32	—61	—459	1.81×10^{-4}
Flaming Gorge	—	—	—496	11.09×10^{-4}
Yellowtail	—12	—38	—345	1.97×10^{-4}†

Volume change specimens for Hoover, Grand Coulee, Angostura, and Kortes dams were 4 $\times$ 4 $\times$ 40-in. (10 $\times$ 10 $\times$ 100-cm) prisms. Specimens for all other dams tabulated were 4 $\times$ 4 $\times$ 30-in. (10 $\times$ 10 $\times$ 76-cm) prisms.

*18 $\times$ 18-in. (45.7 $\times$ 45.7-cm) specimen, standard correction to age of 60 days. K_q is in cu ft/sq ft/yr/ft (m^3/m^2/yr/m head); it is a relative measure of the flow of water through concrete.

†Preliminary mix investigations.

TABLE 3.7.1 — THERMAL PROPERTIES OF CONCRETE

		British units							Metric units						
			Coefficient of expansion,* $\frac{1}{°F} \times 10^{-6}$							Coefficient of expansion,* $\frac{1}{°C} \times 10^{-6}$					
Structure	Coarse aggregate type	Temperature, °F	1½ in. (3.8 cm) max	4½ in. (11.4 cm) max	Thermal conductivity, $\frac{Btu}{Ft \times hr \times °F}$	Specific heat, $\frac{Btu}{lb \times °F}$	Density, $\frac{lb}{Ft^3}$	Diffusivity, $\frac{Ft^2}{hr}$	Temperature, °C	1½ in. (3.8 cm) max	4½ in. (11.4 cm) max	Thermal conductivity, $\frac{Kcal}{m \times hr \times °C}$	Specific heat, $\frac{Kcal}{kg \times °C}$	Density, $\frac{kg}{m^3}$	Diffusivity, $\frac{m^2}{hr} \times 10^{-3}$
Hoover	Limestone and granite	50 100 150	5.3	4.8	1.70 1.67 1.65	0.212 0.225 0.251	156.0	0.051 0.047 0.042	10 38 66	9.5	8.6	2.53 2.48 2.45	0.212 0.225 0.251	2500	4.7 4.4 3.9
Grand Coulee	Basalt	50 100 150	4.4	4.6	1.08 1.08 1.09	0.219 0.231 0.257	158.1	0.031 0.029 0.027	10 38 66	7.9	8.3	1.61 1.61 1.62	0.219 0.231 0.257	2534	2.9 2.7 2.5
Friant	Quartzite, granite and rhyolite	50 100 150	—	—	1.23 1.23 1.24	0.216 0.230 0.243	153.8	0.037 0.035 0.033	10 38 66	—	—	1.83 1.83 1.84	0.216 0.230 0.243	2465	3.4 3.2 3.1
Shasta	Andesite and slate	50 100 150	—	4.8	1.32 1.31 1.31	0.219 0.233 0.247	156.6	0.039 0.036 0.034	10 38 66	—	8.6	1.96 1.95 1.95	0.219 0.233 0.247	2510	3.6 3.3 3.2
Angostura	Limestone	50 100 150	4.0	—	1.49 1.48 1.46	0.221 0.237 0.252	151.2	0.045 0.041 0.038	10 38 66	7.2	—	2.22 2.20 2.17	0.221 0.237 0.252	2423	4.2 3.8 3.5
Kortes	Granite, gabbros and quartz	50 100 150	5.2	4.5	1.61 1.60 1.59	0.208 0.221 0.234	151.8	0.050 0.047 0.044	10 38 66	9.4	8.1	2.40 2.38 2.36	0.208 0.221 0.234	2433	4.6 4.1 4.1
Hungry Horse	Sandstone	50 100 150	6.2	5.7	1.72 1.71 1.69	0.217 0.232 0.247	150.1	0.053 0.049 0.046	10 38 66	11.2	10.3	2.56 2.54 2.51	0.217 0.232 0.247	2406	4.9 4.6 4.3
Canyon Ferry	Sandstone, metasiltstone, quartzite, and rhyolite	50 100 150	5.4	5.2	1.63 1.61 1.59	0.214 0.224 0.235	151.3	0.050 0.047 0.045	10 38 66	9.7	9.4	2.42 2.40 2.36	0.214 0.224 0.235	2425	4.6 4.4 4.2
Monticello	Sandstone (graywacke), and quartz	50 100 150	5.2	—	1.57 1.55 1.53	0.225 0.237 0.250	153.1	0.046 0.043 0.040	10 38 66	9.4	—	2.34 2.31 2.28	0.225 0.237 0.250	2454	4.3 4.0 3.7
Anchor	Andesite, latite, and limestone	50 100 150	5.6	4.5	1.14 1.14 1.15	0.227 0.242 0.258	149.0	0.034 0.032 0.030	10 38 66	10.1	8.1	1.70 1.70 1.71	0.227 0.242 0.258	2388	3.2 3.0 2.8
Glen Canyon	Limestone, chert, and sandstone	50 100 150	—	—	2.13 2.05 1.97	0.217 0.232 0.247	150.2	0.065 0.059 0.053	10 38 66	—	—	3.17 3.05 2.93	0.217 0.232 0.247	2407	6.0 5.5 4.9
Flaming Gorge	Limestone and sandstone	50 100 150	—	—	1.78 1.75 1.73	0.221 0.234 0.248	150.4	0.054 0.050 0.046	10 38 66	—	—	2.65 2.60 2.57	0.221 0.234 0.248	2411	5.0 4.6 4.3
Yellowtail	Limestone and andesite	50 100 150	—	4.3	1.55 1.52 1.48	0.226 0.239 0.252	152.5	0.045 0.042 0.039	10 38 66	—	7.7	2.31 2.26 2.20	0.226 0.239 0.252	2444	4.2 3.9 3.6

*1½ in. (3.8 cm) max and 4½ in. (11.4 cm) max refer to maximum size of aggregate in concrete.

restrained concrete as a result of shrinkage and insufficient tensile strength. Cracking is a weakening factor that may affect the ability of the concrete to withstand its designed loads and may also detract from durability and appearance. Volume change data for some mass concretes are given in Table 3.5.1.

3.5.2—Drying shrinkage ranges from less than 200 millionths for low slump lean mixes with good quality aggregates to over 1000 millionths for rich mortars or some concretes containing poor quality aggregates and an excessive amount of water. The principal drying shrinkage of hardened concrete is usually occasioned by the drying and shrinking of the cement gel which is formed by hydration of portland cement. The main factors affecting drying shrinkage are the unit water content and aggregate composition. Other factors influence drying shrinkage principally as they influence the total amount of water in the mix. The addition of pozzolans generally increases drying shrinkage except where the water requirement is significantly reduced, such as with fly ash. Some aggregates, notably graywacke, have been known to contribute to extremely high drying shrinkage.

3.5.3—Autogenous volume change results from the chemical reactions within the concrete. Unlike drying shrinkage it is unrelated to the amount of water in the mix. The net autogenous volume change of most concretes is a shrinkage of from 0 to 150 millionths. When autogenous expansion occurs it usually takes place within the first 30 days after placing. Concretes containing pozzolans usually have greater autogenous shrinkage than portland cement concrete without pozzolans.

3.5.4—The thermal coefficient of expansion of a concrete varies mainly with the type and amount of coarse aggregate in the concrete. Various mineral aggregates may range in thermal coefficients from below 2 millionths to above 8 millionths per deg F. Neat cement pastes will vary from about 6 millionths to 12 millionths depending on the chemical composition and the degree of hydration. The thermal coefficient of the concrete usually reflects the weighted average of the various constituents. Coefficient of expansion tests are frequently conducted on concrete that has been wet screened to 1½ in. (38 mm) maximum-size aggregate in order to work with smaller size specimens. However, the disproportionately larger amount of cement paste which has a higher coefficient results in values higher than that of the mass concrete.

3.5.5—Volume changes can also result from chemical reactions between reactive constituents in the aggregate and the alkalies (Na_2O and K_2O) in the cement and also between soluble sulfates occurring in the soil or water in contact with a concrete structure and the tricalcium aluminate (C_3A) compound in the cement. These volume changes result in deterioration of the concrete and should be avoided. Low-alkali cement should be specified when reactive aggregates are present and cement low in tricalcium aluminate specified when the concrete is exposed to sulfate waters.

TABLE 3.8.1 — SHEAR PROPERTIES OF CONCRETE*

Dam	Age, days	w/c	Compressive strength		Shear strength		Tan ϕ	$\frac{S}{C}$†
			psi	kg/cm²	psi	kg/cm²		
Grand Coulee	28	0.52	5250	369.08	1170	82.25	0.90	0.223
	28	0.58	4530	318.46	1020	71.71	0.89	0.225
	28	0.64	3810	267.84	830	58.35	0.92	0.218
	90	0.58	4750	333.93	1010	71.00	0.97	0.212
	112	0.58	4920	345.88	980	68.89	1.05	0.199
	365	0.58	8500	597.55	1880	132.16	0.91	0.221
Hungry Horse	104	0.55‡	2250	158.18	500	35.15	0.90	0.222
	144	0.55‡	3040	213.71	680	47.80	0.89	0.224
	622	0.60‡	1750	123.02	400	28.12	0.86	0.229
Monticello	28	0.62‡	2800	196.84	610	42.88	0.93	0.218
	40	0.62‡	4120	289.64	950	66.78	0.85	0.231
Shasta	28	0.50	5740	403.52	1140	80.14	1.05	0.199
	28	0.60	4920	345.88	1060	74.52	0.95	0.215
	90	0.50	5450	383.14	1090	76.63	1.05	0.200
	90	0.50	6590	463.28	1360	95.61	1.01	0.206
	90	0.60	5000	351.50	1040	73.11	1.00	0.208
	245	0.50	6120	430.24	1230	86.47	1.04	0.201

*6 × 12 in. (15.2 × 30.5 cm) test specimens — dry. 1½ in. (3.8 cm) maximum-size aggregate.

‡ $\frac{W}{C+P}$

†Shear strength divided by compressive strength.

3.6 — Permeability

3.6.1—Concrete is inherently pervious to water. However, with properly proportioned mixes that are well compacted by vibration, permeability is not a serious problem. Permeability of concrete increases with increasing water-cement ratios. Therefore, low water-cement ratio and good consolidation are the most important factors in producing concrete with low permeability. Air-entraining agents permit the same workability with reduced water content and therefore contribute to reduced permeability. Pozzolans usually reduce the permeability of the concrete. Permeability coefficients for some mass concretes are given in Table 3.5.1.

3.7 — Thermal properties

3.7.1—Thermal properties of concrete are significant in connection with keeping differential volume change at a minimum in mass concrete, extracting excess heat from the concrete, and dealing with similar operations involving heat transfer. These properties are specific heat, conductivity, and diffusivity. The main factor affecting the thermal properties of a concrete is the mineralogic composition of the aggregate. Since the selection of the aggregate to be used is based on other considerations little or no control can be exercised over the thermal properties of the concrete and tests for thermal properties are conducted only for providing constants to be used in behavior studies as described in Chapter 5. Specification requirements for cement, pozzolan, percent sand, and water content are modifying factors but with negligible effect. Entrained air is an insulator and reduces thermal conductivity but other considerations which govern the use of the entrained air outweigh the significance of its effect on thermal properties. Although the thermal properties of the concrete are largely dependent on the mineralogic composition of the aggregate, an aggregate such as granite, for example, can have a rather wide range of thermal properties depending on the source of the aggregate. Quartz aggregate is particularly noted for its high value of thermal conductivity. Thermal property values for some mass concretes are given in Table 3.7.1. Thermal coefficient of expansion is discussed in Section 3.5.5 under volume change.

3.8 — Shear properties

3.8.1—Shear properties for concretes containing 1½ in. (3.8 cm) maximum-size aggregates are listed in Table 3.8.1. These include compressive strength, shear strength, and coefficient of internal friction (tan ϕ) which are related linear functions determined from results of triaxial tests. Linear analysis of triaxial results gives a shear strength slightly above the true value.

3.8.2—The shear strength relationships reported are linearly analyzed using the Mohr envelope equation $Y = C + X \tan \phi$ in which C (unit cohesive strength) is defined as the shear strength at zero normal stress. Tan ϕ, slope of the line, represents the coefficient of internal friction. X and Y are normal and shear stresses, respectively. In many cases, the shear strengths in Table 3.8.1 were higher for specimens of greater age; however, no definite trend is in evidence. Shear strength varied from 0.20 to 0.23 of the compressive strength for the various concretes shown.

3.9 — Durability

3.9.1—A durable concrete is one which will withstand the effects of service conditions to which it will be subjected, such as weathering, chemical action, and wear. Numerous laboratory tests have been devised for measuring durability of concrete, but it is extremely difficult to obtain a direct correlation between laboratory tests and field service.

3.9.2 *Weathering resistance*—Disintegration of concrete by weathering is caused mainly by the disruptive action of freezing and thawing and by expansion and contraction, under restraint, resulting from temperature variations and alternate wetting and drying. Entrained air improves the resistance of concrete to damage from frost action and should be specified for all concrete subject to cycles of freezing and thawing. Selection of good materials, use of entrained air, low water-cement ratio, proper proportioning, and placement to provide a watertight structure usually provide a concrete that has excellent resistance to weathering action.

3.9.3 *Resistance to deterioration from chemical attack*—Chemical attack can occur from: (1) chemical reactions between constituents of the concrete, (2) exposure to acid waters, (3) exposure to sulfate bearing waters, and (4) leaching by mineral-free waters. In mass concretes usually only the first of these presents a serious problem. Chemical reactions between alkalies in cement and some mineral constituents of aggregates are characterized by excessive expansion and cracking of the concrete. Where it is necessary to use an aggregate containing reactive constituents, low-alkali cement should be specified. Also as further insurance against alkali-aggregate reaction a suitable pozzolan should be specified.

No type of portland cement concrete is very resistant to attack by acids. Should this type of exposure occur the concrete is best protected by surface coatings.

Sulfate attack can be rapid and severe. The sulfates react chemically with the hydrated lime and hydrated tricalcium aluminate in cement paste to form calcium sulfate and calcium sulfoaluminate; these reactions are accompanied by considerable expansion and disruptions of the concrete. Concrete containing cement low in tricalcium aluminate is more resistant to attack by sulfates.

Hydrated lime is one of the products formed when cement and water combine in concrete. This product is readily dissolved in pure water which may occur in high mountain streams. Surfaces of tunnel linings, retaining walls, piers, and other structures are often disfigured by lime deposits from water seeping through cracks, joints, and interconnected voids. With dense, impermeable concrete leaching is seldom severe enough to impair the serviceability of the structure.

3.9.4 *Resistance to erosion*—The principal causes of erosion of concrete surfaces are cavitation and the movement of abrasive material due to flowing water. Use of concrete of increased strength and wear resistance offers some relief but the best solution lies in the prevention, elimination, or reduction of the causes by proper design, construction, and operation of the concrete structure.

CHAPTER 4 — CONSTRUCTION

4.1 — Batching

4.1.1.—Proper batching of mass concrete requires little that is different from the accurate, uniform, reliable batching that is essential for other classes of concrete.[34,35]

However, because efficient mixes for mass concrete contain unusually low portions of cementing materials, sand, and water, the critical level of workability of these mixes is more sensitive to shortcomings in the uniformity of batching. Fortunately, there are several factors which tend to compensate for these requirements. Foremost among these is the fact that usually production of mass concrete is on a larger scale, particularly where it is used in dams, and it is therefore economically more feasible than it often is on smaller jobs to specify and use the most effective methods and equipment. Foremost among these are: (1) finish screening of coarse aggregate at the batching plant, preferably on horizontally operating screens; (2) refinements in batching equipment such as full scale springless dials which register all stages of the weighing operation; (3) automatic weighing and cutoff features; (4) interlocks to prevent recharging when some material remains in a scale hopper; (5) a device for instant reading of approximate moisture content of sand; and (6) graphic recording of the various weighing and mixing operations. In large central plant mixers, the large batches commonly used for mass concrete also tend to minimize the effect of variations.

4.1.2—Since greater use is made in mass concrete of such special purpose ingredients as ice, air-entraining agents, water-reducing and set-controlling admixtures, and fly ash or other pozzolans, the dependable batching of these materials has become a very important aspect of the batching facilities. For most efficient use of ice, its temperature must be less than 32 F (0 C); it must be brittle-hard, dry, and finely broken. Such ice is best batched by weighing from a well insulated storage bin, and if quickly discharged thereafter with the aggregates, it will achieve its full potential for reducing the temperature of the concrete. Pozzolan is batched much the same as cement.

4.1.3—Liquid admixtures are generally batched by volume, although weighing equipment has also been used successfully. Reliable admixture batching equipment is available from some admixture or batch plant manufacturers. Batching accuracy of volumetric batchers should be within ±3 percent of the amount required or ±1 fluid ounce (30 cc), whichever is greater. When batching by weight, accuracy should be ±3 percent of that required. Means should be provided so that a visual accuracy check may be made. Provisions should be made to prevent batching of admixture while discharge valve is open. Interlocks should also be provided that will prevent inadvertent extra or overdose of the admixture. Particularly with air-entraining and water-reducing admixtures, any irregularities in batching can cause troublesome variation with slump and/or air control. The use of comparatively dilute solutions reduces gumming in the equipment. For continuing good operation the equipment must be maintained and kept clean. The use of timed-flow systems is discouraged. Also it is important to provide winter protection for storage tanks and related delivery lines.

4.2 — Mixing

4.2.1—To quickly discharge concrete with a slump less than 2 in. (5 cm), and to ensure even distribution of the larger coarse aggregate in the concrete as discharged, mixers for mass concrete are limited to stationary, central plant mixers. Commonly these mix a 4-cu yd (3-m^3) batch although good work has been done with 2- (1.5 m^3) and 8-cu yd (6-m^3) mixers. The quick and uniform discharge features are accomplished by a tilting discharge arrangement. Paving mixers and truck mixers are not as well suited for mass concrete. There is no record of the performance of turbine-type mixers for mass concrete of low slump, lean mix, and aggregate larger than 3 in. (7½ cm) except for one European dam where 4-in. (10-cm) aggregate was reportedly used. Turbine-type mixers have been successfully used for mass concrete containing 3-in. aggregate.

4.2.2—Specifications for mixing time range from a minimum of 1 min for the first cubic yard plus 15 sec for each additional cubic yard of mixer capacity[34] to 1½ min for the first yard, or the first 2 yd, plus 30 sec for each additional yard of capacity.[35] Blending the materials during batching makes it possible to reduce the mixing period. Some of the mixing water and aggregate should lead other materials into the mixer to prevent sticking and clogging. Specifications usually state that mixing times must be lengthened or may be shortened, depending on the results of mixer performance tests. Criteria for these are found in ASTM C 94, Table I.[36] Mixing time is best controlled by a timing device which will not release the discharge mechanism until the time set for it has expired.

4.2.3—During mixing the last opportunity exists to obtain a batch that has the desired uniform consistency or slump. This requires alertness and attentiveness on the part of the inspector and operator but the operator must have and must use the necessary facilities for this purpose. Preferably the operator should be stationed in the plant where he can see the batch in the mixer and be able to judge whether its slump is correct. If the slump is low, perhaps due to suddenly drier aggregate, he can immediately compensate with a little more water and maintain the desired slump. Lacking this arrangement to see into the mixer, he should be able to see the batch as it is discharged. From this he can note any change from former batches and make water adjustments accordingly. A sand moisture meter will assist in arriving at the appropriate quantitative adjustment.

4.3 — Placing

4.3.1—Placing includes preparation of horizontal construction joints, transportation, handling, placement, and vibration of the concrete.[17,34,35,37,38]

4.3.2—There are various methods for cleanup of horizontal joint surfaces preparatory to placement of the next lift, including green cutting, sandblasting, high pressure water jet, and the use of surface retarders. Under some but by no means all circumstances, each of these methods may do a fairly acceptable job.* However, to be certain of a first class, unquestionably clean and satisfactory surface every time under varying circumstances, sandblasting, preferably wet to avoid dust hazard, is required just prior to concrete placement. Actually, for the high caliber of results it provides, the cost of wet sandblasting is little, if any, more than the cost of trying to obtain a high quality surface by other means. The less effective methods frequently have to give way to sandblasting at the final inspection before starting the next lift of concrete.

4.3.3—Brooming a thin layer of sand-cement mortar on horizontal construction joints has long been standard before placing mass concrete. Often this is brushed into and mixed with puddles of water on a surface that is too wet. Mortar batches delivered at intervals during the placement as the fresh concrete face advances across the lift surface tend to interfere with a smooth running batching, mixing and placing operation. In spite of these possible problems the mortar coat is considered by some agencies as good insurance for bond under normal job conditions.

Tests conducted by the U. S. Army Engineer Waterways Experiment Station[39] failed to establish superiority of bond or watertightness by use or exclusion of a mortar coat. For mortar of the same water-cement ratio as the concrete, superior joints were obtained without the mortar coat on joints sandblasted at 2 days of age and covered after 3 days age. Mortar of slightly lower water-cement ratio than the concrete was beneficial on joints green-cut with air-water jet, cured 14 days and air dried 13 days before placing the second lift. Other tests indicated no definite superiority of joints made with or without mortar. Dry surface conditions proved superior on surfaces covered within 28 days; however no advantages could be detected for dry surfaces subjected to 62 days of air drying prior to treatment. To ensure a tight, invisible joint (as shown by drill cores), it is imperative that the first layer of concrete be very thoroughly and systematically vibrated to full depth and that any rock clusters at batch perimeters where buckets are dumped, are scattered.

4.3.4—Mass concrete for dams is transported in buckets which may range from 2 to 12 cu yd (1.5 to 9 m^3) in capacity. Railcars, trucks, cableways or cranes, or some combination of these, may be used to deliver the buckets to the point of placement. As a rule, a bucket size of 4 to 8 cu yd (3 to 6 m^3) is preferable, since smaller buckets often do not discharge readily and each delivery is too small to organize well into the placement scheme. On the other hand, the 12-yd (9-m^3) bucket puts such a large pile in one place

*Of methods other than sandblast, the high pressure jet of not less than 6000 psi (420 kg/cm^2) is showing the most reliable performance for normal joint intervals and mass concrete mixes. It has the advantage of involving no cleanup and disposal of used sandblast sand or supplying and handling it initially.

that much of the crew's time is devoted to vibrating and spreading. Extra care must be taken to offset the possibility of insufficient vibration at depth in the center or around the perimeter contacts. Usually, mass concrete of proper mix proportions and low slump does not segregate during such transportation over the relatively short distances usually involved.

4.3.5—Mass concrete is best placed in successive layers. These layers should not exceed 18 to 20 in. (45 to 50 cm) in thickness for mass concrete with 4 to 6 in. (10 to 15 cm) maximum-size aggregate and less than 1½-in. (4-cm) slump, placed with 4 to 8-cu yd (3 to 6-m^3) buckets and powerful 6 in. (15 cm) diameter vibrators. The layers should not exceed 12 to 15 in. (30 to 38 cm) in thickness for mass concrete with 3 to 4 in. (7.6 to 10 cm) maximum-size aggregate and less than 2-in. (5-cm) slump, placed with smaller buckets and less powerful vibrators. Shallower rather than deeper layers give better assurance of satisfactory consolidation and freedom from rock pockets at joint lines, corners, and other form faces, as well as within the block itself.

The layer thicknesses should be an even fraction of the lift height or of the depth of the block. (One-third of a 5-ft lift is 20 in.; one-fifth of a 7.5-ft lift is 18 in.) The layers are carried forward and added in the block by means of successive rows of bucket dumps so there will be a setback of about 5 ft (1.5 m) between the forward edges of successive layers. Placement of the steps is organized so as to expose a minimum of surface to lessen warming of the concrete in daytime warm weather and reduce the area affected by rain in wet weather. This minimizes the effort that may be necessary to offset these effects.[38] A greater setback than 5 ft unnecessarily exposes cold concrete to heat gain in warm weather and, in rainy weather, increases the danger of water damage; a narrower setback will cause concrete above it to sag when the step is vibrated afterwards to make it monolithic with the succession of adjacent forward batches placed later against that step. This stepped front progresses forward until the block forms are filled.

4.3.6—Vibration is the key to successful use of efficient, lean, low-slump, large MSA mass concrete. In recent years in the United States, vibration has for the most part been done by large one-man, air-driven, spud-type vibrators. In Europe and Japan a battery of large vibrators is operated on the front of a track-mounted tractor, but concrete must be placed (and exposed) in horizontal layers over the entire block. Ample and effective vibration equipment is available; anything less than this should not be tolerated. Ineffectual equipment is more costly to the builder because of a slower placing rate and the hazard of poor consolidation. Specific recommendations for mass concrete vibration are given in the report of ACI Committee 609.[51]

It is impossible to overemphasize that vibration of each batch must be systematic and should thoroughly cover and deeply penetrate the batch. Particular attention must be paid to ensure full vibration where the perimeters of two batches join, since the outer edge of the first batch is not vibrated (lest it flatten and pull away) until the next batch is placed against it. Then the two can be vibrated monolithically together without causing either edge to flow downward. To ensure penetration for several inches into lower layers, vibrators are operated in a vertical position and should remain in operation at each penetration point until large air bubbles have ceased to rise and escape from the concrete. The average time for one vibrator to fully consolidate a cubic yard of this concrete may be as much as 1 min. Overvibration of low-slump mass concrete is unlikely. To simplify cleanup operations, the top of the uppermost layer should be leveled and made reasonably even by means of vibration. Large aggregate should be all but embedded and boards should be laid on the surface in sufficient number to prevent deep footprints.

4.4 — Curing

4.4.1—Mass concrete is best cured with water for the additional cooling benefit in warm weather. In cold weather, probably little curing is needed beyond the moisture provided to keep the concrete from drying during its initial protection from freezing, but it should not be saturated when it is exposed to freezing. In above-freezing weather when moisture is likely to be lost from its surfaces, mass concrete should be water cured for at least 14 days or up to twice this time if pozzolan is used as one of the cementing materials. Except when insulation is required in cold weather, surfaces of horizontal construction joints should be kept moist until new concrete is placed on them or until the wetting will no longer provide beneficial cooling. Sealing-compound curing is not the best method of curing mass concrete but in some instances is the most practical. If used on construction joints it must be completely removed by sandblasting or imperfect bond will result.

4.5 — Forms

4.5.1—Forms for mass concrete have the same basic requirements for strength, mortar-tightness under vibration, accuracy of position, and generally good surface condition as those described in *Formwork for Concrete,* (ACI SP-4).[40] In United States practice they differ somewhat from other formwork because of the comparatively low height normally required of each lift. There may be some increase of form pressures due to use of low temperature concrete and the impact of dumping large buckets of concrete near the forms, despite the relieving effect of the generally low slump of mass concrete. Form pressures actually depend on the methods used in placing concrete next to the form. For this reason some designers use 80-90 percent of equivalent hydrostatic pressure plus 25 percent for impact when control of field conditions is questionable.

Form ties to wire loop anchors in the previous lift and braces have long been used. Many large jobs are now equipped with forms supported by cantilever strongbacks anchored firmly into the lift below. Some of these are given the additional support of form ties, particularly when the concrete is low in early strength. These forms are raised by mechanized A-frames and considerable care is necessary to avoid spalling concrete around the anchor bolts in the low-early-strength concrete of the lift being stripped. These are bolts which will be used to hold the forms from moving outward in the next form setup.

High lift concrete formwork of the type used in Canada is comparable to that for structural concrete except that ties may be 20 to 40 ft (6 to 12m) long rather than 20 to 40 in. (50 to 100 cm). To use large aggregate concrete, widely spaced large diameter high tensile ties are required to permit passage of concrete buckets.

4.5.2—To mask offsets in nonoverflow sections that sometimes occur at horizontal joint lines, and to generally dress up and improve appearance of formed surfaces, it has been found that a beveled grade-strip and 1 in. (2½ cm) or larger triangular toe fillet at the top and bottom of the forms can be used to create an effective and pleasing groove which serves these purposes. A 1-in. chamfer should also be used in the corners of the forms at the upstream and downstream end of contraction joints for the sake of appearance, because sharp corners of the blocks otherwise are often damaged and cannot be effectively repaired. Such chamfers also prevent pinching and spalling of joint edges caused by high surface temperatures.

4.5.3—Sloping forms sometimes reach so far over the construction joint that it is difficult to get buckets close enough to place concrete without separation in the toe and to vibrate it well. Accordingly, some specifications require such forms to be hinged so the top half can be held in a vertical position until concrete is placed up to the hinged elevation. The top half is then lowered into position and concrete placing continued.

4.5.4—A common forming problem for spillway sections of gravity dams is encountered in the flatly sloping and curved portions forming the crest and the bucket. These are the slopes that range from horizontal to about 1½ to 1 when use of fixed forms begins. Some builders attempt to shape such slopes the hard way with screed guides and strikeoff. Actually such surfaces are much more easily shaped with temporary holding forms. With no strikeoff involved the regular mass concrete face mix is as readily used as one with small aggregate. All that is required are strong, solidly anchored ribs between which rows of form panels are placed row-on-row upward as the lift space is filled, and removed starting row-on-row at the bottom when the concrete will no longer bulge out of shape but is still responsive to finishing operations. Considerable time and labor are saved by this method and it permits a proper concrete to be used.

4.6 — Height of lifts

4.6.1—From the standpoint of construction, the higher the lift the fewer construction joints; with 7.5-ft (2.3-m) lifts there are only two-thirds as many joints as when 5-ft (1.5-m) lifts are used. From the standpoint of temperature control in cold weather, the shallower the lift, the more heat of hydration will escape before the next lift is placed and the maximum temperature reached will be lower. In hot weather with lean mixes and precooling the reverse may be true. In general, the longer the time between lifts the better for cooling, provided ambient temperatures are lower than those of the concrete surfaces while internal temperature is rising, since a lower ambient temperature will reduce the maximum temperature attained.*

4.6.2—The concern over temperature rise and the specified means to limit it are design considerations which will be reflected in designation of lift height and placing frequency on drawings and in specifications. (Reference is made to Chapter 5.) Influencing factors are size and type of dam or other massive structure which involves concrete properties and cement content, prevailing climate during construction and in service, construction schedule required, and other temperature controls imposed. Accordingly, heights of lift commonly range from 2½ ft (76 cm) for several lifts just above the foundation in some cases, through 5-ft (1.5-m) and 7½-ft (2.3-m) lifts in many dams and other work, to 10 ft (3 m) or

*When lift thickness is increased above 10 ft (3 m), the law of diminishing returns becomes increasingly apparent as losses from the upper surface become a decreasing percentage of the heat generated within the full depth of the lift. Hence, with very deep lifts, the internal temperature does not differ greatly whether long delays are enforced or whether the lifts are stacked in rapid succession. In such extreme cases, continuous placing in high lifts may be preferable, especially as a means of minimizing joint cleanup, or to permit the use of slipforms, e.g., for massive piers.

more in thin arch dams, piers, and abutments, and other semimass concrete structures of such limited horizontal thickness that temperature rise due to cement hydration is not a matter of major concern.

High lift mass concrete construction has been adopted by some authorities, particularly in Canada, in an attempt to reduce potential leak paths and minimize cracking in dams built in cold and even subzero weather. In its extreme form, the method provides for continuous placing of lifts up to 50 ft (15 m) high using wood or insulated forms with housings and steam heat. Under these placing conditions the adiabatic temperature rise of the concrete and the maximum temperature drop to low stable temperatures are approximately equal. For control of cracking most design criteria restrict this maximum drop to 25 or 35 F (14 to 20 C). Design requirements can be met, under these conditions, by controlling, through mix proportioning, the adiabatic rise to these levels.[41] With precooled [50 F (10 C)] mass concrete of low cement content in a warm climate, ambient heat removes the advantage of shallower lifts and is the reason 7½ (2.3 m) or even 10-ft (3-m) lifts have been permitted by specifications on several dam projects in recent years.

4.7 — Cooling and temperature control

4.7.1—Currently it is common practice to precool mass concrete before placement. Efficient equipment is now available to produce such concrete at temperatures less than 50 F (10 C) in practically any summer weather. Merely the use of finely chipped ice instead of mixing water and the shading of damp (but not wet) aggregate will reduce the temperature to a value approaching 50 F (10 C) in all but the hottest weather. By this means the temperature of 4 in. (10 cm) MSA mass concrete in the hot Sacramento Valley in California was held to an average of 51 F (11 C) during August 1962. In hot, humid areas aggregates can be cooled by vacuum and inundation of aggregates. For other recommendations see ACI 605-59.[42]

It has been found that the best uniformity of mix results when maximum use of ice is made for precooling mass concrete. Cooling methods which rely on moisture in the aggregate invariably cause moisture fluctuation in the aggregate as batched, with corresponding detriment to the uniformity of slump. Moreover, systems which involve considerable handling and movement of the coarse aggregate are likely to develop considerable fines which, if the aggregate is moist, will not be removed during the finish screening and will serve only to increase mixing water requirement and reduce strength. If full use of ice is made and temperatures lower than can be obtained by this means are desired, coarse aggregate can usually be cooled sufficiently by passing frigid air through it in the batch plant bins, and sand can be cooled by shading. Sand must also have a uniformly low moisture content.

4.7.2—To obtain full advantage of the low placing temperature, the concrete temperature should not be allowed to rise, due to ambient conditions, higher than it naturally would due to heat of hydration alone in the first few weeks after placement. Preferably, heat should be removed and the surfaces cured as cold as possible.[43] This will reduce the thermal differential tending to crack the surface later when much colder ambient conditions may occur. During placement in warm weather, warming of the cold concrete can be minimized by placing it at night, by managing placement so that minimum areas are exposed, and, if placement must be done in the sun, by fog spraying the work area so that temperatures are at least as low as they are in the shade. Cooling sprays should also be started immediately over completed portions of the block.

4.7.3—Aside from pipe cooling, much can be done during the curing period to prevent heating and to remove heat from the hardening concrete. Specifically the following practices are suggested: (1) steel forms can be used for quick transfer of heat and, when air is warmer than the concrete, the steel forms can be kept sprayed with cold water, cooled with fine evaporating sprays, and shaded until they are removed (See Chapter 5 for conditions favoring insulated forms); (2) water curing and shading of formed and finished surfaces can be conducted in the same manner, designed to cool as well as to provide moisture for curing; and (3) water curing of horizontal construction joints can be arranged with controlled evaporative spraying such that no water remains on the surface long enough to become warm.

4.7.4—Pipe cooling is used to control the rise in concrete temperature in restrained zones near foundations when maximum temperatures cannot be maintained by other, less expensive, cooling measures. It is also normally required to control the minimum opening of contraction joints when grouting of joints is necessary. It consists of a series of evenly spaced pipe coils through which refrigerated or cold water is circulated. The size and spacing of pipes depends on block size, thickness of lift, and amount of heat to be removed. (Examples of design are given in Chapter 5). When precooled [50 F (10 C)] concrete is used pipe cooling is usually not required except in certain lifts immediately above the foundation if placed in warm weather.

4.8 — Grouting contraction joints

4.8.1—"Is this necessary?" might well be asked concerning the grouting of many contraction joints, particularly those in straight or nearly straight gravity dams. With increasingly effective use of cold concrete as placed, and especially when narrow shrinkage slots are left and later filled with cold concrete, it may be questioned whether contraction joint grouting serves much purpose for high, thin, arch dams, since a little downstream cantilever movement will bring the joints into tight contact. Grouting relieves later arch and cantilever stresses and it remains general practice to grout contraction joints in arch dams.

4.8.2—Where there is reason to grout contraction joints, the program of precooling and postcooling should be so arranged as to secure a joint opening of at least 0.025 in. (0.064 cm) to assure complete filling with grout even though, under special test conditions, grout may penetrate much narrower openings.

CHAPTER 5 — BEHAVIOR

5.1 — Thermal stresses and cracking

5.1.1—The most important characteristic of mass concrete that differentiates its behavior from that of structural concrete is its thermal behavior. Mass concrete structures are typically structures having large dimensions. These large dimensions in a material whose thermal properties allow only slow movements of heat, means that heat trapped within a mass concrete structure can hardly escape unless aided artificially. For instance, the laws of heat transfer tell us that heat can escape from a body inversely as the square of its least dimension. Consider a number of walls, made of average concrete and exposed to cooler air on both faces. For a wall 6 in. (15 cm) thick, 95 percent of the heat in the concrete will be lost to the air in 1½ hr. For a 5 ft (1.5 m) thick wall, this same amount of heat would be lost in a week. For a 50 ft (15 m) thick wall, which might represent the thickness of an arch dam, it would take 2 years to dissipate 95 percent of the heat stored, and for a 500 ft (152 m) thick dam, such as Boulder, Shasta, Grand Coulee, and many other massive dams, it would take 200 years to dissipate this amount of heat. Thus in ordinary structural construction most of the heat generated by the hydrating cement is dissipated almost as fast as it is generated and there is little temperature differential from the inside to the outside of the body. Since change of temperature results in change of volume, and when restrained, in change of stress in the tensile direction, very thin structures are relatively free from thermal cracking. However, the temperature rise in mass concrete as discussed in previous chapters, is nearly adiabatic and must be dealt with in mass concrete structures.

5.1.2—In mass concrete, thermal stresses are developed in two ways: from the dissipation of the heat of hydration and from periodic cycles of ambient temperature. Since all cements, as they hydrate, cause concrete to heat up to some degree, it is fortunate that the strength and the corresponding cement requirements for mass concrete are much less than those of normal structural concretes; hence, temperature rise is restricted. As has been described in preceding chapters, some relief in temperature rise can be gained, in addition to the minimal use of cement, by the use of substitutions for cement, and by the use of special types of cement with lower, or delayed heats of hydration. When the potential temperature rise of a concrete has been reduced to the minimum, the temperature drop that causes tensile stress and cracking can be reduced to zero if the initial temperature of the concrete is set below the final stable temperature of the structure by the amount of the potential temperature rise. Economy in construction can be gained if the initial temperature is set slightly above this value so that a slight temperature drop is allowed, such that the tensile stresses built up during this temperature drop are less than the tensile strength of the concrete at that time.

5.1.3—Previous chapters describe methods for reducing the initial temperature of concrete, and the benefits on placing of the use of cold concrete. It can be seen that if the maximum temperature of the concrete is appreciably above that of the final stable temperature of the mass, volume changes will take place continuously in massive structures for centuries. Since this is intolerable in some structures that depend on fast construction for economy, this excess heat must be removed artificially. The usual method is by circulating a cooling medium in embedded pipes.

5.1.4—The behavior of the surface of mass concrete structures is tremendously affected by daily and annual cycles of temperature. At the surface the temperature of concrete responds almost completely to daily variations in air temperature, while 2 ft (60 cm) from the surface, only 10 percent of the daily surface temperature variation is felt in the concrete. The annual temperature cycle, however, affects the concrete at much greater depths. Ten percent of the annual variation in temperature can be felt 25 ft (7.6 m) from the surface. Directing our attention back to the surface, it can be seen that the surface is absolutely defenseless against stress cracking caused by temperature change. Since the interior reacts so much more slowly than the surface to cycles of temperature it is as though the surface were completely restrained by the interior concrete. Thus in a location where the surface temperature varies annually by 100 F (56 C) which is only average, if the concrete is assumed to have a modulus of elasticity as low as 3.0×10^6 psi (0.21×10^6 kg/cm^2), without cracking the surface stresses would vary about 1000 psi (70 kg/cm^2) above and below the average. While concrete can quite easily take 1000 psi (70 kg/cm^2) compression, it usually has nowhere near that capacity in tension, and cracking is inevitable. However, because of the rapid deterioration of the temperature cycles with distance from the surface the variation in stress is likewise dissipated rapidly, with the result that surface cracking due to temperature changes is confined to a relatively shallow region at and near the surface. Thus it can be considered that mass concrete as in dams behaves as though the surface, even though cracked by temperature cycles, protects the structural integrity of the concrete below it.

5.1.5—The above statements about the effect of variations in surface temperature on cracking explain how injudicious form stripping at time of extreme contrast between internal and outside temperatures will inevitably result in surface cracking. This phenomenon has been termed "thermal shock" and will occur when forms that act as insulators are removed on an extremely cold day. Modern steel forms that allow the surface temperature of the concrete to more nearly correspond to that of the air reduce this differential temperature somewhat. However, they are open to the objection that the thermal shock may be felt from extremes of temperature right through the form into the concrete. Either a dead airspace or insulation should be provided to protect concrete surfaces where steel forms are used. Insulation requirements and the age for form stripping to avoid cracking the surface depend on the air temperature and the strength of concrete. For protection requirements see ACI 306-66.[44]

5.1.6—For an average concrete, 1 percent of the annual temperature cycle will be felt 50 ft (15 m) from the surface. Thus for a high arch dam 100 ft (30 m) thick, all of the mass will respond to the annual variation of temperature in different degrees. Considering that any change in temperature will cause corresponding change in stress, it can be seen that the entire thickness of this dam will undergo stress and volume change. Stresses at any particular part of the dam will be the sum of two superimposed types of stress: the structural stress due to the average temperature rise of the entire cross section and the local stress due to the difference between the average temperature rise and the temperature at a particular location. Most arch dams are designed merely for the average temperature change; designers should also consider the effect of the additional stresses caused by the differential temperature between the average and the maximum.

5.2 — Volume change

5.2.1—In Chapter 3, properties affecting volume change have been listed for a number of dams. Before accepting for use in mass concrete the numerical values given for drying shrinkage, autogenous volume change and permeability, it must be remembered that all of these tests were performed on quite small specimens, and except for the permeability tests, none actually contained mass concrete. However, the values given can be used as a guide to the actual behavior of mass concrete in service. First, it can be seen that the permeability of mass concrete is very small, a fraction of a foot per year. As a working guide to the behavior of concrete, it can be considered that mass concrete gives up water with great reluctance, but accepts it at a free surface fairly easily. Thus at a surface exposed to air, the surface is quite capable of drying out, while the concrete behind that surface has lost little, if any, of its moisture content. This leads directly to surface shinkage cracking in mass concrete in two ways. The most common cause of surface shrinkage cracking is due to drying at the surface. It can be seen in the table of properties that the concrete exhibiting the minimum shrinkage had a volume change of roughly 300 millionths, and if this can be considered completely restrained by interior concrete with all its moisture intact and therefore with no shrinkage, surface stresses greater than 1000 psi (70 kg/cm^2) are a natural result. Actually, concrete can withstand nowhere near that tensile stress, and the result is an extensive pattern of surface cracking. Exactly as in the case of thermal cracking at the surface, these cracks will extend inward a short distance and disappear in the region of moisture equilibrium.

5.2.2—Whenever a flat surface of mass concrete is finished as in a dam roadway, a spillway apron surface, or a powerplant floor, care must be taken to avoid the conditions causing "plastic shrinkage cracks." This cracking occurs under extreme drying conditions, when water evaporates from the upper surface of the concrete faster than it reaches the surface by water gain. Even as the concrete is setting, wide cracks appear, in the same pattern as found on a drying mud flat, making ugly scars across the entire finished surface. These can be prevented in extreme drying weather by shading the area of finishing operations, by providing barriers against the movement of the air, by fog spraying, by surface sealing, or by any other means available to prevent surface evaporation.

5.3 — Heat generation

5.3.1—Since the outstanding problem of mass concrete construction is the necessity for controlling the heat entrapped within it as the cement hydrates, a short statement will be given here of the thermal properties and mathematical relationships that enable the engineer to estimate rapidly the degree of temperature control needed for a particular application.

Both the rate and the total adiabatic temperature rise differ among the various types of cement. Fig. 5.3.1 shows adiabatic temperature rise curves for mass concretes containing 376 lb per cu yd (223 kg/m^3) of various types of cement with a 4½ in. (114 mm) maximum size aggregate. Values shown are averaged from a number of tests; individual cements of the same type will vary considerably from the average for that type. As might be expected, high-early-strength cement, Type III, is the fastest heat generator and gives the highest adiabatic temperature rise. Type IV, or low-heat cement, is not only the slowest heat generator, but gives the lowest total temperature rise. Since the cement is the active heat producer in a concrete mix, the temperature rise of concretes with cement contents differing from 376 lb per cu yd (223 kg/m^3) can be estimated closely by multiplying the values shown on the curves by a factor representing the proportion of cement.

5.3.2—When a portion of the cement is replaced by a pozzolan, the temperature rise curves are greatly modified, particularly in the early ages. While the effects of pozzolans differ greatly, depending on the composition and fineness of the pozzolan and cement used in combination, a rule of thumb that has worked fairly well on preliminary computations has been to assume that pozzolan gives off about 50 percent as much heat as the cement that it replaces.

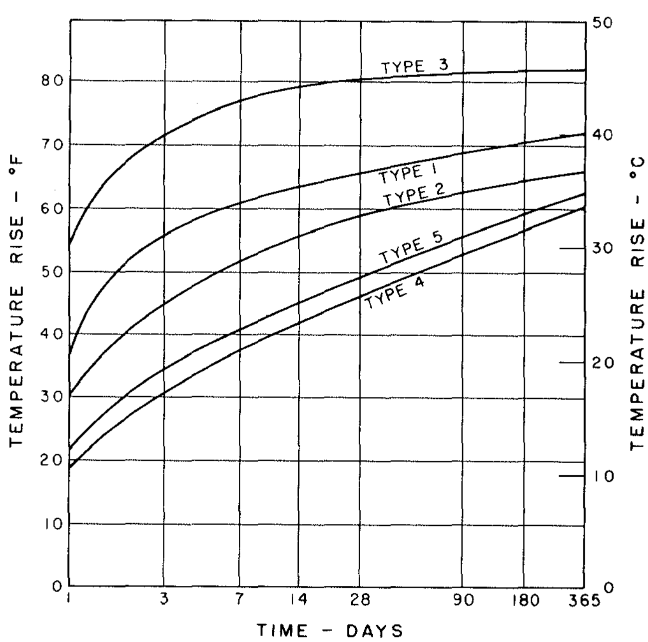

Figure 5.3.1 — Temperature rise of mass concrete

5.3.3—In general, the effects of water-reducing retarders in concrete are felt only during the first few hours after mixing and can be neglected in preliminary computations using these curves. However, in studies involving millions of cubic yards of concrete, as in a dam, the above remarks should be applied only to preliminary computations, and the adiabatic temperature rise of the exact mix to be used in the mass concrete starting at the initial temperature contemplated should be determined.

5.3.4—The characteristic that determines the relative ability of heat to flow through a particular concrete is its thermal diffusivity which is defined as:

$$h^2 = \frac{K}{C\rho}$$

where

h^2 = diffusivity, sq ft per hr (m^2/hr)
K = conductivity, Btu/ft/hr/deg F (Kcal/m/hr/deg C)
C = specific heat, Btu/lb/deg F (Kcal/kg/deg C)
ρ = density of the concrete, lb per cu ft (kg/m^3)

The value of diffusivity is largely affected by the rock type used in the concrete. Table 5.3.4 shows diffusivities for concrete made of a number of rock types.

TABLE 5.3.4 — DIFFUSIVITY AND ROCK TYPE

Coarse aggregate	Diffusivity of concrete, sq ft per day (m^2/day)
Quartzite	1.39 (0.129)
Limestone	1.22 (0.113)
Dolomite	1.20 (0.111)
Granite	1.03 (0.096)
Rhyolite	0.84 (0.078)
Basalt	0.77 (0.072)

The higher the value of diffusivity, the more readily heat will move through the concrete. If the rock type is not known, an average value of diffusivity can be taken as 1.00 sq ft per day (0.093 m^2/day) although as can be seen from the table the value of diffusivity varies substantially from this average value.

5.3.5—Another source of heat in mass concrete is the variation of external temperature.[52] If the external temperature variation can be considered to be expressed as a sine wave, and if, as in a dam, the body of concrete is sufficiently thick so

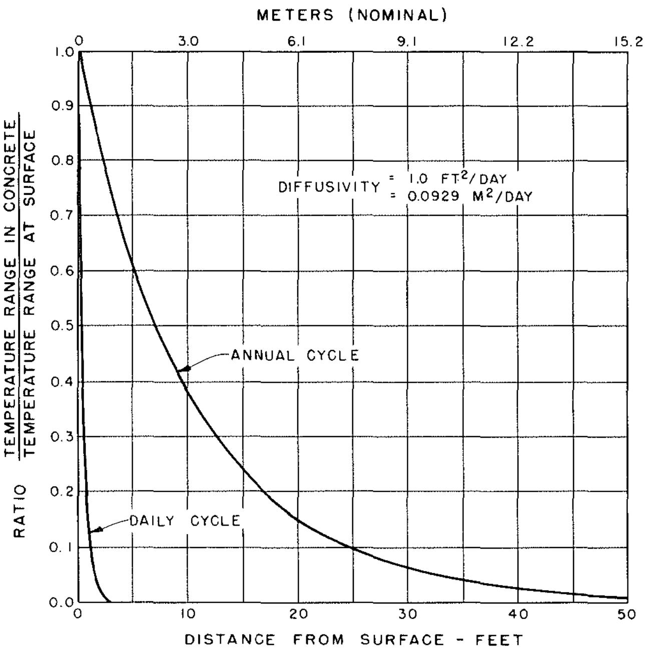

Fig. 5.3.5 — Temperature variation with depth

that the temperature variation is negligible opposite to the exposed face, the range of temperature variation any distance in from the surface can be computed from

$$\frac{R_x}{R_o} = e^{-x\sqrt{\pi/h^2\gamma}}$$

where

R_x is the range at distance x from the surface,
R_o is the range at the surface ($x = 0$)
h^2 is the diffusion constant
γ is the period of the cycle of temperature variation in days

For an average concrete with a diffusivity of 1 sq ft per day (0.093 m²/day) the penetration of the daily and the annual temperature cycles is as shown in Fig. 5.3.5.

5.4 — Heat dissipation

5.4.1—Studies of the dissipation of heat from bodies of mass concrete can be accomplished by the use of charts and graphs, or by direct computation.

When the body to be analyzed can be readily approximated by a known geometrical shape, charts are available for the direct determination of heat losses. For instance, Fig. 5.4.1 can be used to determine the loss of heat in hollow and solid cylinders, slabs with one or two faces exposed, or solid spheres. The application of the values found on these graphs can easily be made to a wide variety of problems such as the cooling of dams or thick slabs of concrete, the cooling of concrete aggregates, artificial cooling of mass concrete by use of embedded pipes, and the cooling of bridge piers. The following five examples are typical concrete cooling problems which can be solved by use of Fig. 5.4.1. In these examples and Fig. 5.4.1, the following notation is followed:

t = time in days
h^2 = diffusivity, sq ft per day
D = thickness of concrete section, ft
θ_o = initial temperature difference between concrete and ambient material
θ_m = final temperature difference between concrete and ambient material

Example 1

At a certain elevation an arch dam is 70 ft thick and has a mean temperature of 100 F. If exposed to air at 65 F, how long will it take to cool to 70 F? Assume $h^2 = 1.20$ sq ft per day.

Initial temperature difference, $\theta_o = 100 - 65 = 35$ F
Final temperature difference, $\theta_m = 70 - 65 = 5$ F
The portion of the original heat remaining is

$$\frac{\theta_m}{\theta_o} = \frac{5}{35} = 0.142$$

From Fig. 5.4.1, using the slab curve

$$\frac{h^2t}{D^2} = 0.18$$

Then

$$t = \frac{0.18\,D^2}{h^2} = \frac{0.18\,(70)^2}{1.20} = 735 \text{ days}$$

Example 2

A mass concrete bridge pier has a cross section of 25 × 50 ft, and is at a mean temperature of 80 F. Determine the mean temperature at various times up to 200 days if the pier is exposed to water at 40 F and if the diffusivity is 0.90 sq ft per day. For a prismatic body such as this pier, the part of original heat remaining may be computed by finding the part remaining in two slabs of respective thickness equal to the dimensions of the pier, and multiplying the two quantities so obtained to get the total heat remaining in the pier. For this two-dimensional use, it is better to find for various times the heat losses associated with each direction and then combine them to find the total heat loss of the pier.

Initial temperature difference, $\theta_o = 80 - 40 = 40$ F
For the 25-ft dimension

$$\frac{h^2t}{D^2} = \frac{0.90 \times t}{(25)^2} = 0.00144t$$

and for the 50-ft dimension

$$\frac{h^2t}{D^2} = \frac{0.90 \times t}{(50)^2} = 0.00036t$$

Then calculate numerical values of $0.00144t$ and $0.00036t$ for times from 10 to 200 days. See Table 5.4.1. These values can be used with Fig. 5.4.1 to obtain the θ_m/θ_o ratios for both 25-ft and 50-ft slabs. The product of these ratios indicates the heat remaining in the pier, and can be used to calculate the final temperature difference θ_m. θ_m values are added to the temperature of surrounding water to obtain mean pier temperatures at various times up to 200 days.

TABLE 5.4.1—EXAMPLE 2 CALCULATIONS

Time, days	0.00144t	0.00036t	$\left(\frac{\theta_m}{\theta_o}\right)$ 25	$\left(\frac{\theta_m}{\theta_o}\right)$ 50	$\left(\frac{\theta_m}{\theta_o}\right)$ pier	θ_m	Temperature, deg F
10	0.0144	0.0036	0.73	0.87	0.64	26	66
20	0.0288	0.0072	0.61	0.80	0.49	20	60
30	0.0432	0.0108	0.53	0.77	0.41	16	56
40	0.0576	0.0144	0.46	0.73	0.34	14	54
60	0.0864	0.0216	0.35	0.67	0.23	9	49
100	0.144	0.036	0.19	0.57	0.11	4	44
200	0.288	0.072	0.05	0.40	0.02	1	41

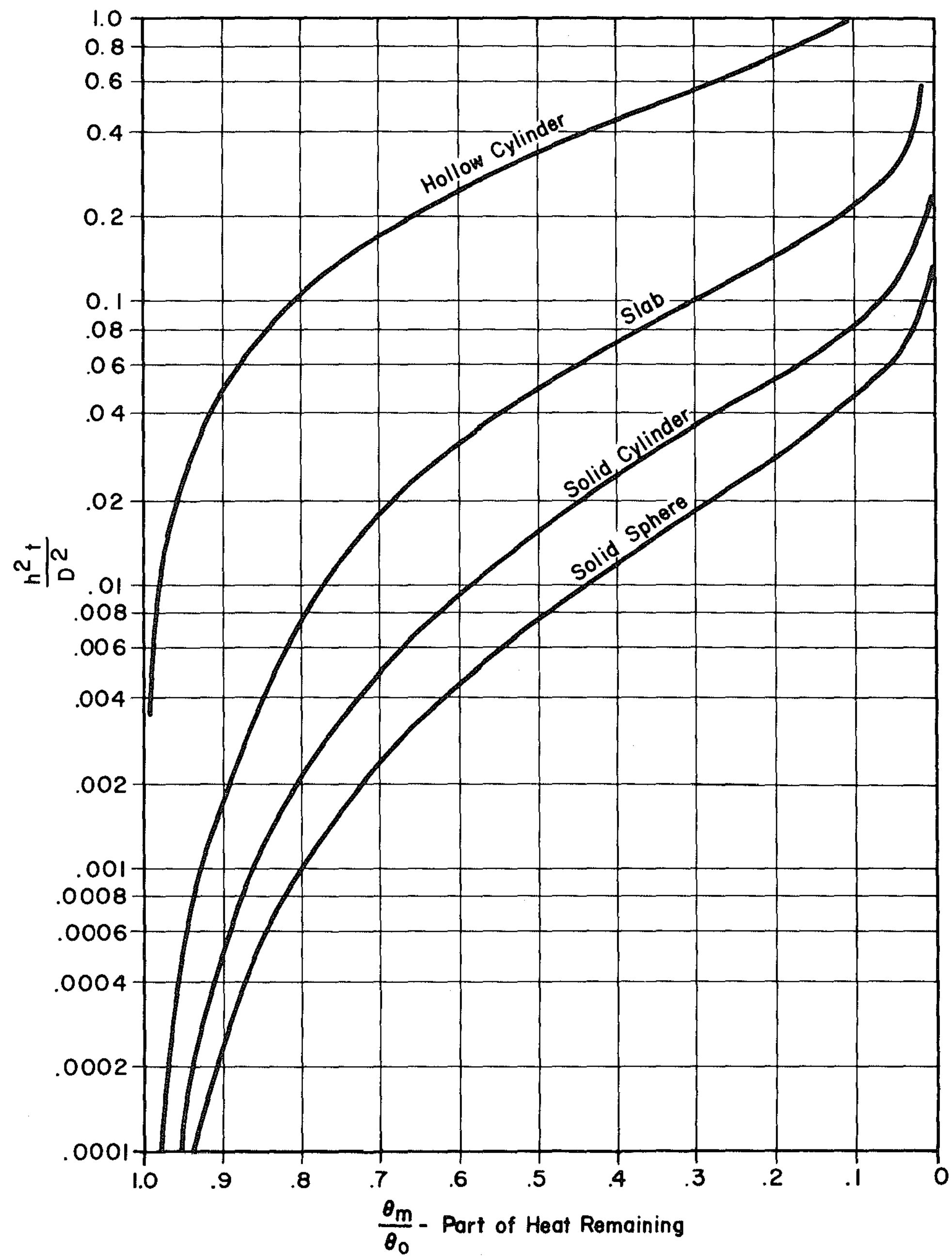

Fig. 5.4.1 — Heat loss from solid bodies

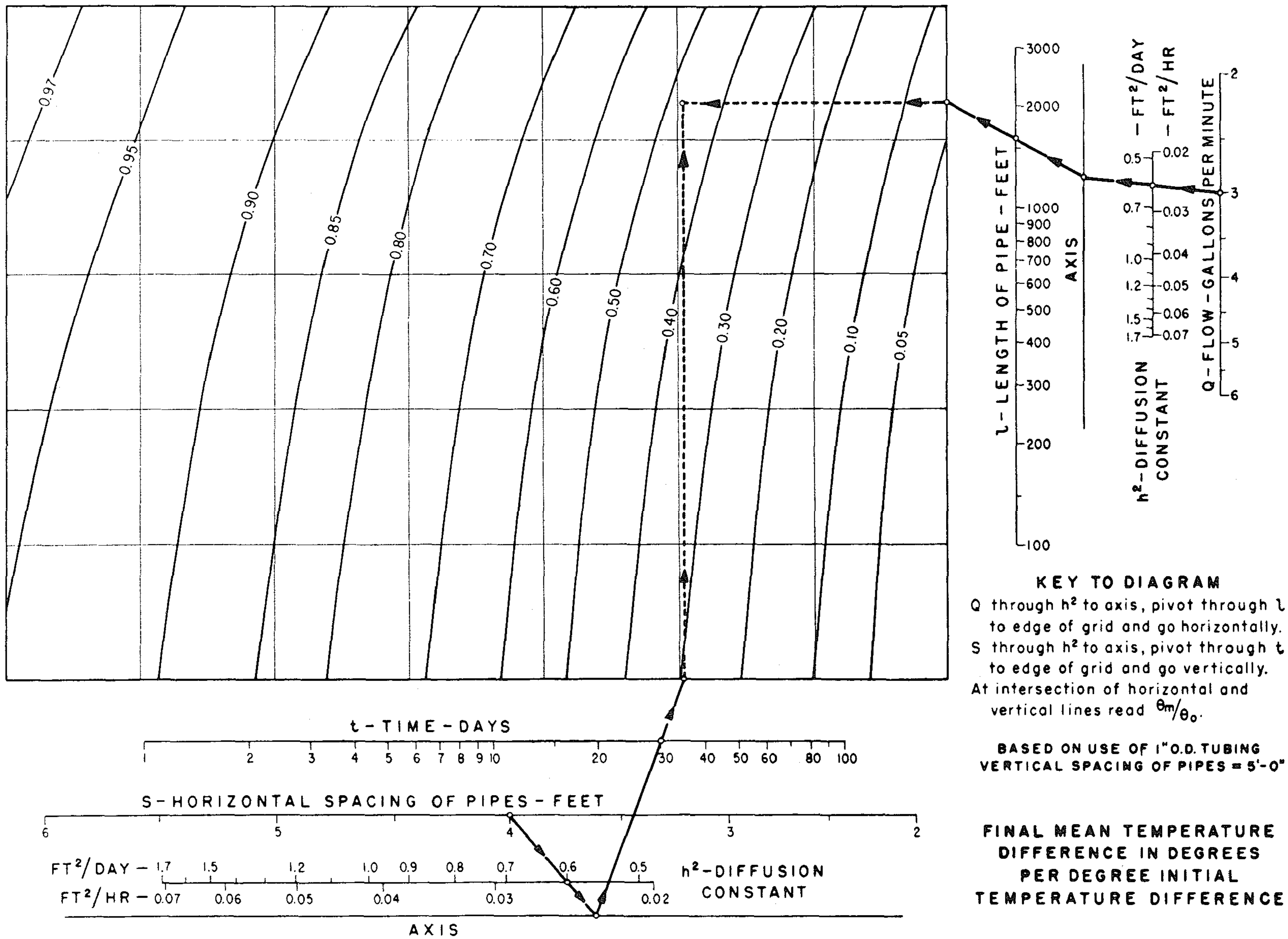

Fig. 5.4.2(a)—Mean temperature of concrete (from Reference 46). (Note: 1.0 ft equals 0.305 m and 1.0 gal. per min equals 0.0631 liters per sec)

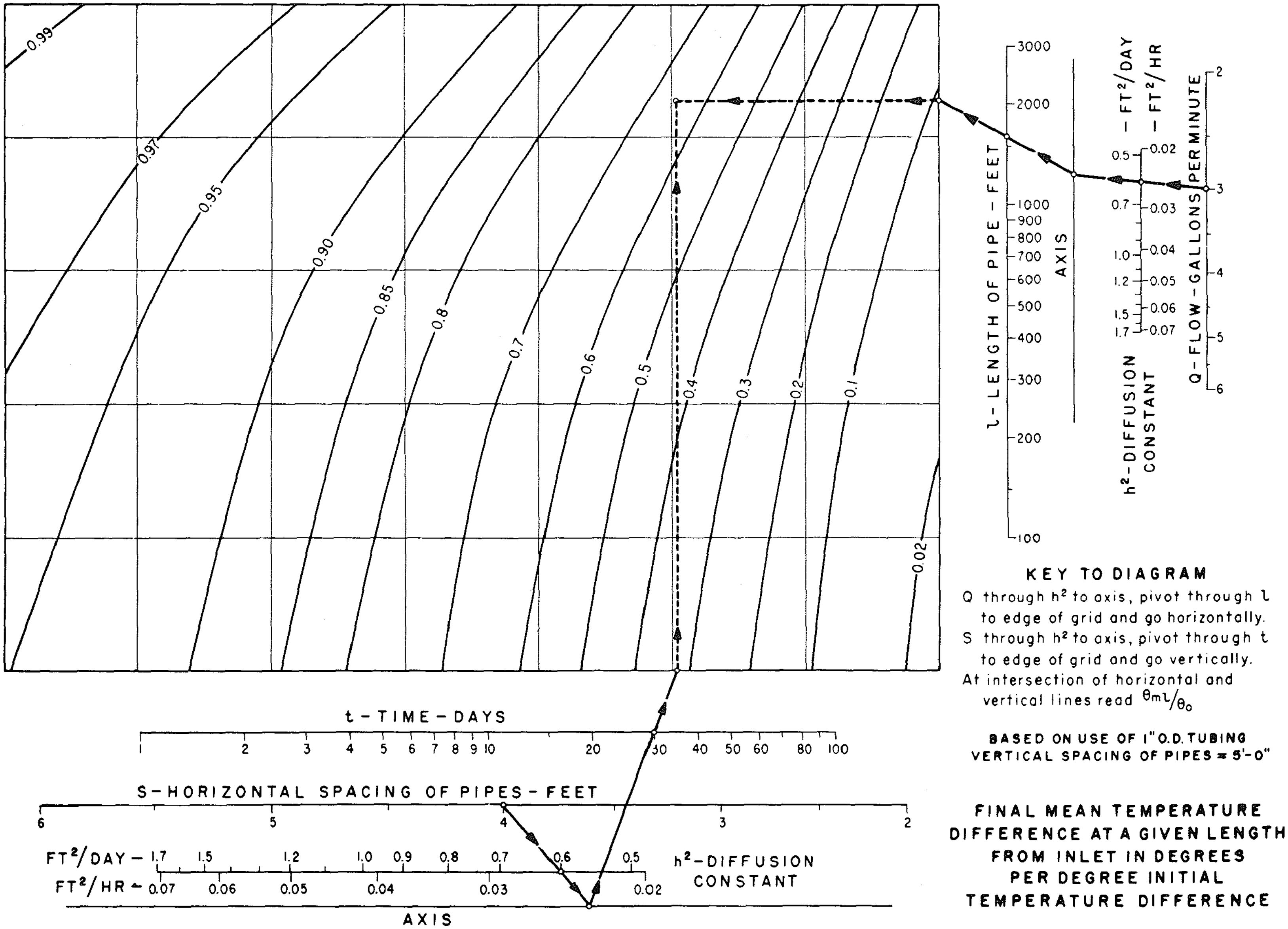

Fig. 5.4.2(b)—Mean temperature of concrete at given length from inlet (from Reference 46). (Note: 1.0 ft equals 0.305 m and 1.0 gal. per min equals 0.631 liters per sec)

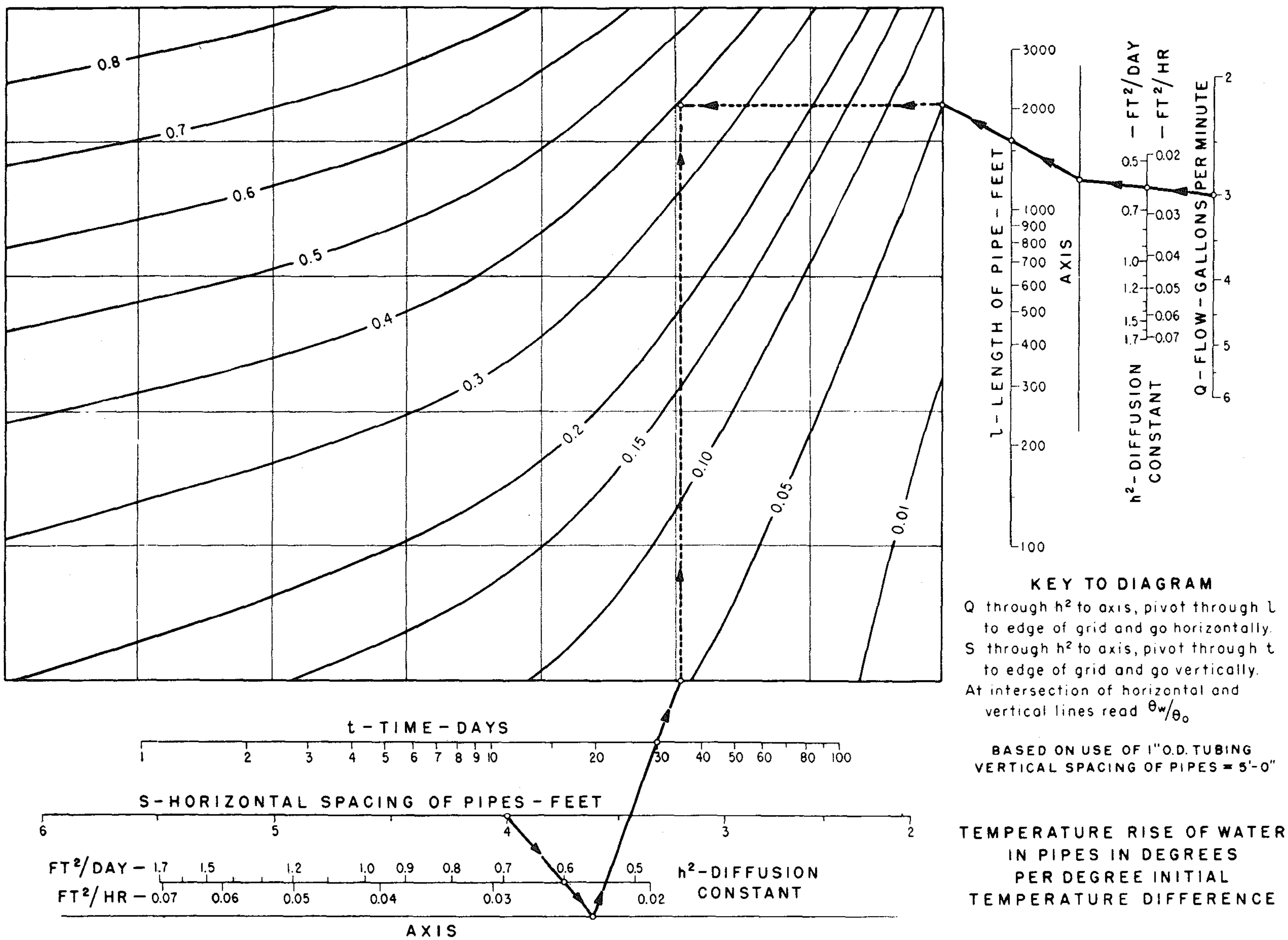

Fig. 5.4.2(c)—Temperature rise of water in cooling coils (from Reference 46). (Note: 1.0 ft equals 0.305 m and 1.0 gal. per min equals 0.0631 liters per sec)

Example 3

Granite aggregate at an initial temperature of 90 F is to be chilled in 35 F water in making precooled mass concrete. The largest particles can be approximated as 6 in. diameter spheres. How long must the aggregate be immersed to bring its mean temperature to 40 F?

For granite, $h^2 = 1.03$ sq ft per day

Initial temperature difference, $\theta_o = 90 - 35 = 55$ F

Final temperature difference, $\theta_m = 40 - 35 = 5$ F

$$\frac{\theta_m}{\theta_o} = \frac{5}{55} = 0.091$$

From Fig. 5.4.1, for the above value

$$\frac{h^2t}{D^2} = 0.05$$

$$t = \frac{0.05 \times (6/12)^2}{1.03} = \frac{0.05 \times 0.25}{1.03} = 0.0121 \text{ days}$$

or approximately 17 min.

Example 4

A 50 ft diameter circular tunnel is to be plugged with mass concrete with diffusivity of 1.20 sq ft per day. The maximum mean temperature in the concrete is 110 F, and the surrounding rock is at 65 F.

Without artificial cooling, how long will it take for the temperature in the plug to reach 70 F, assuming the rock remains at 65 F?

Initial temperature difference, $\theta_o = 110 - 65 = 45$ F

Final temperature difference, $\theta_m = 70 - 65 = 5$ F

$$\frac{\theta_m}{\theta_o} = \frac{5}{45} = 0.11$$

From Fig. 5.4.1,

$$\frac{h^2t}{D^2} = 0.08$$

$$t = \frac{0.08 \times (50)^2}{1.20} = 167 \text{ days}$$

Example 5

A closure block of concrete initially at 105 F is to be cooled to 45 F to provide a joint opening of 0.025 in. prior to grouting contraction joints. How long will it take to cool the mass by circulating water at 38 F through cooling pipes spaced 4 ft 6 in. horizontally and 5 ft 0 in. vertically. Assume concrete to be made with granite aggregate.

Cross sections handled by one pipe are $4.5 \times 5.0 = 22.5$ sq ft.

Equivalent cylindrical diameter is $22.5 = \pi D^2/4$

$$D^2 = \frac{4 \times 22.5}{\pi} = 28.7 \text{ sq ft}$$

$$D = 5.35 \text{ ft.}$$

Initial temperature difference, $\theta_o = 105 - 38 = 67$ F

Final temperature difference, $\theta_m = 45 - 38 = 7$ F

$$\frac{\theta_m}{\theta_o} = \frac{7}{67} = 0.104$$

Refer to Fig. 5.4.1, using the curve for the "hollow" cylinder since cooling is from within cross section. For the calculated value of θ_m/θ_o

$$\frac{h^2t}{D^2} = 1.0$$

$$t = \frac{1.0 \times 28.7}{1.03} = 27.9 \text{ days}$$

About the same results can be achieved with greater economy if the natural cold water of the river is used for part of the cooling. Control of the rate of cooling must be exercised to prevent thermal shock and in many cases postcooling is conducted in two stages.

Assume river water is available at 60 F, cool to 68 F, and then switch to refrigerated water at 38 F. How much time will be taken in each operation, and what is total cooling time?

Initial cooling, $\theta_o = 105 - 60 = 45$ F, $\theta_m = 68 - 60 = 8$ F

$$\frac{\theta_m}{\theta_o} = \frac{8}{45} = 0.178, \quad \frac{h^2t}{D^2} = 0.82$$

$$t = \frac{0.82 \times 28.7}{1.03} = 22.9 \text{ days}$$

Final cooling, $\theta_o = 68 - 38 = 30$ F, $\theta_m = 45 - 38 = 7$ F

$$\frac{\theta_m}{\theta_o} = \frac{7}{30} = 0.233$$

$$\frac{h^2t}{D^2} = 0.70$$

$$t = \frac{0.70 \times 28.7}{1.03} = 19.6 \text{ days}$$

Total time is $19.6 + 22.9 = 42.5$ days, but of this, the time for using refrigeration has been cut by one-third.

5.4.2—Fig. 5.4.2(a) through 5.4.2(c) can be used for the determination of all the characteristics of an artificial cooling system for mass concrete. Fig. 5.4.2(a) can be used for the determination of the actual cooling accomplished in a given number of days with a given pipe spacing and flow of coolant. Fig. 5.4.2(b) gives more detail on the cooling of the mass concrete by determining the temperature at various points along the length of the cooling coil, and Fig. 5.4.2(c) can be used to determine the temperature rise of the coolant in the coil.

For instance, the results of Example 5 can be obtained with less trouble from Fig. 5.4.2(a), making further assumptions that the cooling coil is 200 ft long, and the coolant is circulated at the rate of 5 gpm.

As before, $\theta_m/\theta_o = 0.104$, $t = 35$ days.

The difference in results is in the fact that the mean takes into account the variation in temperature of the cooling water along the pipe as it extracts heat from the concrete. (If a longer pipe is assumed, say 600 ft, the time increases to almost 40 days.)

5.4.3—However, all the foregoing methods are only approximations; in the usual case hydration and cooling go on simultaneously. For this more general case in which it is necessary to determine actual temperature gradients, Schmidt's method[45] has proved of immense value. The concept and application is so simple that it can be performed quite easily with a desk calculator, and yet for complicated cases can easily be programmed for computer application. Without going into its derivation, Schmidt's method is based on the theorem that if the body under question is con-

TABLE 5.4.3 — TEMPERATURES THROUGH TWO LIFTS

Distance above ground	Station											
	T 0	T 0.5	T 1		T 1.5		T 2		T 2.5		T 3	
	$\Delta\theta = 20$		$\Delta\theta = 11$		$\Delta\theta = 6$		$\Delta\theta = 3$		20/2.5		11/20	
12								0	0	0		
11								0			10	21
10								0	0	20		
9								0			20	31
8								0	0	20		
7								0			20.4	31.4
6	0				0	0		0	9.5	20.7		
5	0	20	10	21			16	19			25.4	27.4
4	0	20			26	32			27.6	30.1		
3	0	20	20	31			33.2	36.2			32.7	34.7
2	0	20			28.5	34.5			32.8	35.3		
1	0	20	15	26			26.5	29.5			28.2	30.2
0	0	10			15.5	18.5			20.0	21.2		
−1	0	0	5	5			10.5				13.5	
−2	0	0			2.5	2.5			5.8			
−3		0	0	0			1.2				3.2	
−4						0			0.6			
−5							0				0.3	
−6									0			
											0	

Note that in the above computation two steps are required to produce the temperature at the end of the half-day period: the first step averages the adjacent temperatures, and the second step adds the adiabatic temperature rise of the concrete.

sidered to be divided into a number of equal elements, and if a number of physical limitations are satisfied simultaneously, the temperature for a given increment at the end of an interval of time is the average of the temperature of the two neighboring elements at the beginning of that time interval. The necessary physical relationship is:

$$\Delta t = \frac{(\Delta x)^2}{2h^2}$$

where Δt is the time interval, Δx is the length of element, and h^2 is the diffusion constant. Units of Δt and Δx must be consistent with units in which h^2 is expressed. Stated mathematically, θ_p, θ_q, and θ_r are the temperatures of three successive elements at time t, then at time t_2

$$\theta_q + \Delta\theta_q = 1/2\,(\theta_p + \theta_r)$$

The beauty of Schmidt's method is that it can be extended to cases of two-dimensional and three-dimensional heat flow. For the two-dimensional case the numerical constant 2 is replaced by 4, and the averaging must take into account temperatures on four sides of the given element. For the three-dimensional case, the constant 2 is replaced by the number 6 and the averaging must be carried on for six elements surrounding the cubic element in question. The following example demonstrates the use of Schmidt's method in a practical problem.

Determine temperatures throughout two 6-ft lifts of mass concrete placed at 2-day intervals, using a concrete mix containing 376 lb of Type II cement per cu yd (223 kg/m^3). Since the method uses temperature differences rather than actual temperatures, assume air temperature and concrete placement temperature to be 0 F, and take h^2 to be 1.00 sq ft per day. If space interval is 1.0 ft, then time interval is

$$\Delta t = \frac{(\Delta x)^2}{2h^2} = \frac{1}{2 \times 1} = ½ \text{ day.}$$

In the following table the adiabatic temperature rise in ½ day intervals for a 3-day investigation is taken from Fig. 5.3.1. (An estimate of the rise of 1/2 day was obtained by projecting the Type II curve back.) The change in temperature $\Delta\theta$ is determined by subtracting the temperature at any time interval from that of the preceding time interval.

Time, days	Adiabatic temperature rise, deg F	$\Delta\theta$
0	0	
0.5	20	20
1	31	11
1.5	37	6
2	40	3
2.5	42.5	2.5
3	44.5	2.0

In the tabular solution of Table 5.4.3 the space interval of 1.0 ft divides each lift into six elements or stations. Boundaries such as rock surface, construction joints, and exposed surfaces must be clearly defined. The adiabatic temperature rise at the rock surface is just one-half of the concrete rise since the rock is not generating heat. At a construction joint the rise is the average of the two lifts which are generating heat at different rates at any given time. At the exposed surface the adiabatic rise is zero since the heat is dissipated as quickly as it is generated from the concrete below. Normally where there are several stations in each lift the temperature distribution within the lift at any given time can be obtained with sufficient accuracy by calculating only half of the points at any one time as shown.

5.5 — Instrumentation

5.5.1—During the past 40 years, very sophisticated instrumentation has been developed for the measurement of the behavior of structures made of mass concrete. Instrumentation has been installed for two primary reasons: for a continued check on the safety of the structure, and for new knowledge on the behavior of mass concrete under service conditions.

5.5.2—The first measurements of structural behavior of mass concrete structures were made by extensometer measurements at the surface, possibly influenced by laboratory measurements on models. It has been shown in the foregoing paragraphs that excessive stress and cracking at the surface resulted in certain failure of such measurements. Thus, the only meaningful measurements of strain and stress in mass concrete must come from instruments embedded well within the mass, or at least far enough from the free surface to avoid the effects of daily temperature cycles. Since the instruments must be embedded, they have become nearly standardized as electrical measuring instruments, connected by cables to convenient reading points in galleries or at the surface of mass concrete structures. Electrical instruments are now available for measuring strain, stress, temperature, joint movements, foundation deformations, pore pressure, and the detection of cracks. Additional instruments have been devised for measuring gross movements such as deflection, tilting, and directional changes.

5.5.3—Before describing the instruments themselves, it would be well at this point to review the physical conditions under which they must work. The instruments must be rugged enough to be embedded in fresh mass concrete, using care, of course. When measuring strain and displacements, in particular, the instruments must be at least three times the length of the largest particle in the fresh concrete mix. Since they are electrical in nature, they must not only be waterproof, but all materials must be resistant to the aggressive solutions of the hydrating cement.

5.5.4—After brief experimentation with other types of instruments, measurements in mass concrete in United States practice have become nearly standardized in Carlson instruments read through rubber covered electrical cables. These comprise strain meters, stress meters, joint meters, deformation meters, pore pressure cells, and reinforcement meters. In each of these devices, two sets of unbonded steel wires are so arranged that when subjected to the action to be measured, one set increases in tension, while the other decreases. A test set is available that records the electric characteristics, resistance, and resistance-ratio of these elastic wires, from which can be determined through the use of calibration constants, the temperature at the point and the characteristic quantity sought. These instruments can be installed in the fresh concrete, have proved durable in service, have a constant zero reading, maintain their calibration, and as now constructed may be depended on for 25 years. For reliability, instruments must not only be installed in sufficiently large groups to give all values necessary according to theories of the mechanics of solids but account must be taken of the many things that can happen during and after construction to instruments and cable to cause faulty readings. On this account a judicious number of duplicate instruments is necessary. In addition, isolated unusual indications are always suspect and instrumentation must be installed so that readings from different types of instruments tend to reinforce each other and give certainty in the results.

5.5.5—When these precautions are taken, backed up by laboratory determinations of the essential elastic and creep properties of the concrete in question, a clear picture is given of the structural behavior of the mass concrete structure, leading to more accurate analyses of proposed designs. At the same time, indications are available to see how well the service behavior of a structure falls within the assumptions used in its design. Thus, continued assurances may be gained of the satisfactory behavior of the mass concrete structure.

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This report was submitted to letter ballot of the committee which consists of 18 members; 15 members returned ballots, all of whom voted affirmatively.

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Effect of Restraint, Volume Change, and Reinforcement on Cracking of Massive Concrete

Reported by ACI Committee 207

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This report presents a discussion of the effects of heat generation and volume change on the design and behavior of massive reinforced concrete elements and structures. Particular emphasis is placed on the effects of restraint on cracking and the effects of controlled placing temperatures, concrete strength requirements, type and fineness of cement on volume change. Formulas are presented for determining the amounts of reinforcing steel needed to control the size and spacing of cracks to specified limits under varying conditions of restraint and volume change.

Keywords: adiabatic conditions; age; cement types; concrete dams; concrete slabs; cooling; crack propagation; crack width and spacing; **cracking (fracturing)**; creep properties; drying shrinkage; foundations; heat of hydration; heat transfer; machine bases; **mass concrete**; modulus of elasticity; moisture content; placing; portland cement physical properties; portland cements; pozzolans; reinforced concrete; **reinforcing steels**; **restraints**; shrinkage; stresses; structural design; temperature; temperature rise (in concrete); tensile strength; thermal expansion; **volume change**; walls.

CONTENTS

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NOTATION

A = effective tension area of concrete surrounding the reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars, sq in.

A_B = area of a member undergoing volume change

A_b = area of reinforcing bar

A_g = gross area of concrete cross section

A_F = area of foundation or other element restraining shortening of element to be reinforced

A_s = area of steel for a given width

A_s' = area of steel per ft of length for a given width

B, b = width of cross section

C = weight of portland cement per cu yd of concrete, lb

C_{eq} = weight of portland cement plus a percentage of the weight of pozzolan per cu yd of concrete, lb

C_T = linear thermal coefficient, 5×10^{-6} per F for limestone aggregate, 6×10^{-6} per F for siliceous river gravel aggregate

d = depth of member from compressive face to the center of gravity of the reinforcement

d_c = thickness of concrete cover measured from the concrete surface at which cracks are being considered to the center of the nearest reinforcing bar

d_s = assumed depth of tensile stress block for internal restraint considerations

e = the eccentricity of a load with respect to the center of gravity of the section

E_c = modulus of elasticity of concrete

E_F = modulus of elasticity of foundation or restraining element

E_s = modulus of elasticity of steel

F = weight of fly ash per cu yd of concrete, lb

f_c' = specified compressive strength of concrete, psi

f_s = calculated stress in reinforcement, psi

f_t' = tensile strength of concrete, psi

H = perpendicular distance from restrained edge to free edge. Where a slab is subject to edge restraint on two opposite edges, H is one-half the distance between edges

H_a = adiabatic temperature rise of the concrete

h = height of vertical restraining element, column or wall, above fixed base or elemental height of a wall

h^2 = diffusivity in sq ft per day

I_c = moment of inertia of gross concrete section subjected to flexure by the restraining forces

j = ratio of distance between centroid of compression and centroid of tension to the depth, d

K_c = stiffness of vertical restraining element subjected to flexure by the restraining forces

K_f = stiffness of floor system being tensioned by restraint

K_R = degree of restraint. Ratio of actual stress resulting from volume change to the stress which would result if completely restrained. In most calculations it is convenient to use the ratio of the difference in free length change and actual length change to the free length change

k = ratio of depth of compressive area to total depth of flexural member using the straight line theory of stress distribution

L = distance between contraction or expansion joints in the direction of restraint or overall length of a member undergoing volume change

L' = calculated average distance between cracks

N_B = number of reinforcing bars spaced along the B face

N_H = number of reinforcing bars spaced along the H face or faces

n = ratio of modulus of elasticity of steel to that of concrete

p = area of steel divided by the appropriate area of concrete

M_{RH} = the restraining moment to be supplied by the stress reinforcing steel for full height cracking

M_{Rh} = same as above for partial height

R = ratio of the distance from the neutral axis to the tension face of a flexural member to the distance from the neutral axis to the tension steel. Where flexure is not involved $R = 1$

S = surface area of a concrete member exposed to air

T_A = the average minimum ambient air temperature over a prolonged exposure period of 1 week

T_C = the temperature generated by the total quantity of cementitious materials if all were portland cement

T_{C+F} = the temperature generated by the mixture of portland cement and fly ash

T_E = effective temperature change in members including an equivalent temperature change to compensate for drying shrinkage

T_{DS} = an equivalent temperature drop to be used in lieu of drying shrinkage

T_M = the temperature of earth or rock mass

T_p = the placing temperature of the fresh concrete

T_{PE} = the effective placing temperature after accounting for heat gained from or lost to the air

T_1 = high temperature in a temperature gradient

T_2 = low temperature in a temperature gradient

V = volume of a concrete member

W_u = the water content of the fresh concrete per cu yd, lb

w = maximum statistical surface crack width, in.

w = weight of concrete, lb per cu ft, Section 2.3.2

CHAPTER 1—INTRODUCTION

1.1—Scope

This report presents a detailed discussion of the effects of heat generation and volume changes on the design and behavior of massive reinforced concrete elements and structures. It is written primarily to provide guidance for the selection of concrete materials, mix requirements, reinforcement requirements, and construction procedures necessary to control the size and spacing of cracks. Particular emphasis is placed on the effect of restraint to volume change in both preventing and causing cracking and the need for controlling peak concrete temperatures. The quality of concrete for resistance to weathering is not emphasized in recommending reduced cement contents; however, it should be understood that the concrete should be made sufficiently durable to resist expected service conditions apart from strength requirements. The report can be applied to normal structural concrete; however, its application is not usually warranted and the major emphasis is on massive concrete.

1.2—Definition

Mass concrete is defined in "Mass Concrete for Dams and Other Massive Structures"[1] as:

> "Any large volume of cast-in-place concrete with dimensions large enough to require that measures be taken to cope with the generation of heat and attendant volume change to minimize cracking."

Massive reinforced concrete differs from unreinforced mass concrete in that reinforcement is utilized to limit crack widths that may be caused by external forces or by inadequately controlled volume changes.

1.3—Approaches to control of cracking

All concrete elements and structures are subject to volume change in varying degrees, dependent upon the makeup configuration and environment of the concrete. Uniform volume change will not produce cracking if the element or structure is relatively free to change volume in all directions. This is rarely the case for massive concrete members; however, since size alone usually causes nonuniform change and there is often sufficient restraint either internally or externally to produce cracking.

The measures used to control cracking are dependent to a large extent on the economics of the situation and the seriousness of cracking if not controlled. The appearance of cracks in almost any concrete structure is undesirable irrespective of whether or not the designer has allowed for cracks in his design. For this reason some agencies or organizations take the position that cracks should be controlled to the minimum practicable size in all structures. Even so, the economics of construction must be considered. The change in volume can be minimized by such measures as reducing cement content, replacing part of the cement by pozzolans, precooling, postcooling, insulation to control the rate of heat absorbed or lost, and by other measures outlined in Reference 1. Restraint is modified by contraction or expansion joints and also by the rate at which volume change takes place. Construction joints may also be used to reduce the number of uncontrolled cracks which may otherwise be expected. It is usually possible by appropriate consideration of the above measures to eliminate cracks or at least reduce their size. The subject of crack control in mass concrete is also discussed in Chapter 7 of *Control of Cracking in Concrete Structures* by ACI Committee 224.[13]

In the design of reinforced concrete structures, cracking is presumed in the proportioning of reinforcement for stress considerations. For this reason the designer does not normally distinguish between cracks due to volume change and those due to flexure. Instead of employing many of the above recommended measures for controlling volume change, he may choose to add sufficient reinforcing steel to distribute the cracking so that one large crack is replaced by many smaller cracks of acceptable width. The selection of the necessary amount and spacing of reinforcement to accomplish this depends on the extent of the volume change to be expected, the spacing or number of cracks which would occur without the reinforcement, and the ability of reinforcement to distribute cracks.

The degree to which the designer will either reduce volume changes or use steel for the control of cracks in a given structure depends largely on the massiveness of the structure itself and on the magnitude of forces restraining volume change. No clear-cut line can be drawn to establish the extent to which measures should be taken to control the change in volume. Design strength requirements, placing restrictions, and the environment itself are sometimes so severe as to make it impractical to prevent cracking by measures to minimize volume change. On the other hand, the designer normally has a wide range of choices in the selection of design strengths and structural dimensions.

In many cases, the cost of increased structural dimensions by the selection of lower strength concrete, within the limits of durability requirements, is more than repaid by the savings in reinforcing steel, reduced placing costs, and the savings in material cost of the concrete itself. (See Example 6.1.)

CHAPTER 2—RESTRAINT

2.1—General

The term restrain is defined as "to hold back from action; check; suppress; curb; to limit; restrict." Restraint thus acts to limit the change in dimensions and produces strain, with corresponding stress in a concrete member. Numerically, the strain is equal to the product of the degree of restraint existing at the point in question and the change in unit length which would occur if the concrete were not restrained.

All concrete elements are strained to some degree by volume because there is always some restraint provided either by the supporting elements or by different parts of the element itself. Restrained volume change can induce tensile, compressive, or flexural stresses in the elements depending on the type of restraint and whether or not the change in volume is an increase or decrease. We are normally not concerned with restraint conditions which induce compressive stresses in concrete because of the ability of concrete to withstand compression. We are primarily concerned with restraint conditions which induce tensile stresses in concrete which can lead to cracking.

In the discussion which follows, the types of restraint to be considered are external restraint (continuous and discontinuous) and internal restraint. Both types are interrelated and usually exist to some degree in all concrete elements.

2.2—Continuous external restraint

Continuous restraint exists along the contact surface of concrete and any material against which the concrete has been cast. The degree of restraint depends primarily on the relative dimensions, strength, and modulus of elasticity of the concrete and restraining material.

2.2.1 *Stress distribution*—By definition the stress at any point in an uncracked concrete member is proportional to the strain in the concrete. The horizontal stress in a member continuously restrained at its base and subject to an otherwise uniform horizontal length change varies from point to point in accordance with the variation in degree of restraint throughout the member. The distribution of restraint varies with the length to height ratio of the member. For the case of concrete placed without time lapses for lifts, it is shown graphically in Fig. 2.1 which was derived from test data reported in 1940 by Carlson and Reading.[2,2a]

Using the degree of restraint in Fig. 2.1 as K_R the tensile stress at any point on the centerline due to a decrease in length can be calculated from:

$$f_t = K_R \Delta_c E_c \tag{2.1}$$

where

K_R is the degree of restraint expressed as a ratio with 1.0 = 100 percent.

where

Δ_c is the contraction if there were no restraint.

where

E_c is the sustained modulus of elasticity of the concrete at the time when Δ_c occurred and for the duration involved.

The stresses in concrete due to restraint decrease in direct proportion to the decrease in stiffness of the restraining foundation material. The multipliers to be used in determining K_R from Fig. 2.1 is given by:

$$\text{Multiplier} = \frac{1}{1 + \dfrac{A_g E_c}{A_F E_F}}$$

where

A_g = cross sectional area of stressed concrete
A_F = area of restraining mass
E_F = modulus of elasticity of restraining mass, psi

For mass concrete on rock the maximum effective restraining mass area (A_F) can be assumed at $2.5A_g$ and the values of the multipliers are then shown in the following table:

TABLE OF MODIFIERS FOR FOUNDATION RIGIDITY

$\frac{E_F}{E_c}$	Multipliers
∞	1.0
2	0.83
1	0.71
0.5	0.56
0.2	0.33
0.1	0.20

where:

E_F = the modulus of elasticity of the restraining mass.

2.2.2 *Cracking pattern*—When stress in the concrete due to restrained volume change exceeds the tensile strength of the concrete, a crack will form. If a concrete member has a uniform tendency to contract but is restrained at its base or at an edge,

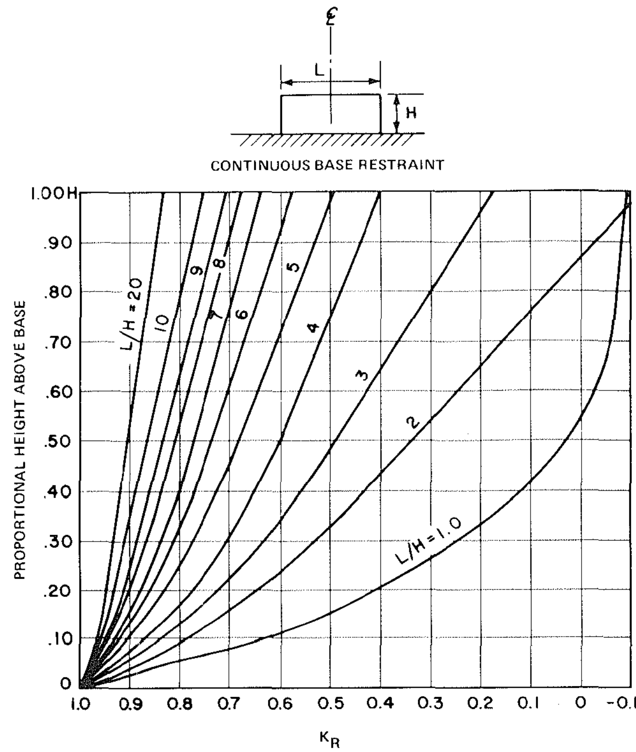


Fig. 2.1—Degree of tensile restraint at center section

cracking will initiate at the base or restrained edge where the restraint is greatest and progress upward or outward until a point is reached where the stress is insufficient to continue the crack. However, if the wall is of limited height, the occurrence of a crack to appreciable height will increase the tensile stress above the end of the crack and very often cause the crack to extend through the entire block. The reason for this is that after the occurrence of the crack, none of the tension caused by the restraint is supported in the region of the crack but must be supported by the uncracked concrete. For L/H ratios greater than about 2.5, Fig. 2.1 of this report and Fig. 155 of Reference 2 indicate that if there is enough tensile stress to initiate a crack, it should propagate to the full block height because of the stress-raising feature just mentioned. Besides, it has been found from many tests that once begun, a crack will continue with half the stress field necessary to initiate a crack.

From the above discussion, unreinforced concrete walls or slabs, subject to base restraint, will ultimately attain cracks through the full block height spaced in the neighborhood of 1.0 to 2.0 times the height of the block. As each crack forms the propagation of that crack to the full height of the block will cause a redistribution of base restraint such that each portion of the wall or slab will act as an individual section between cracks. Using Eq. (2.1) and K_R values from Fig. 2.1 to determine the stress distribution at the base centerline, the existing restraining force and moment at

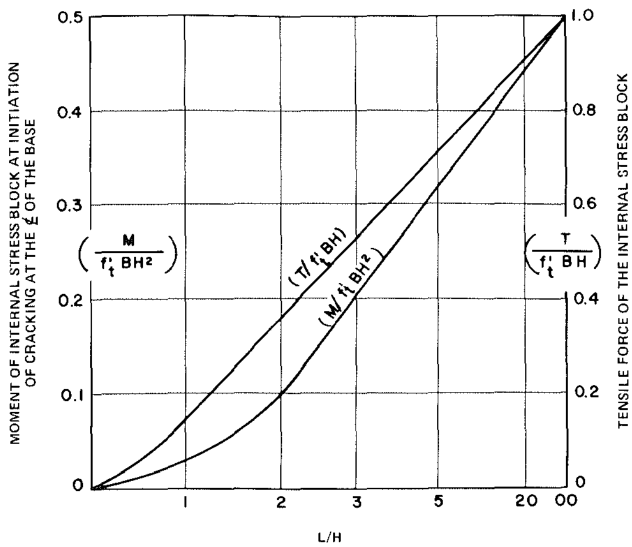


Fig. 2.2—Internal restraint forces at initiation of cracks at the base

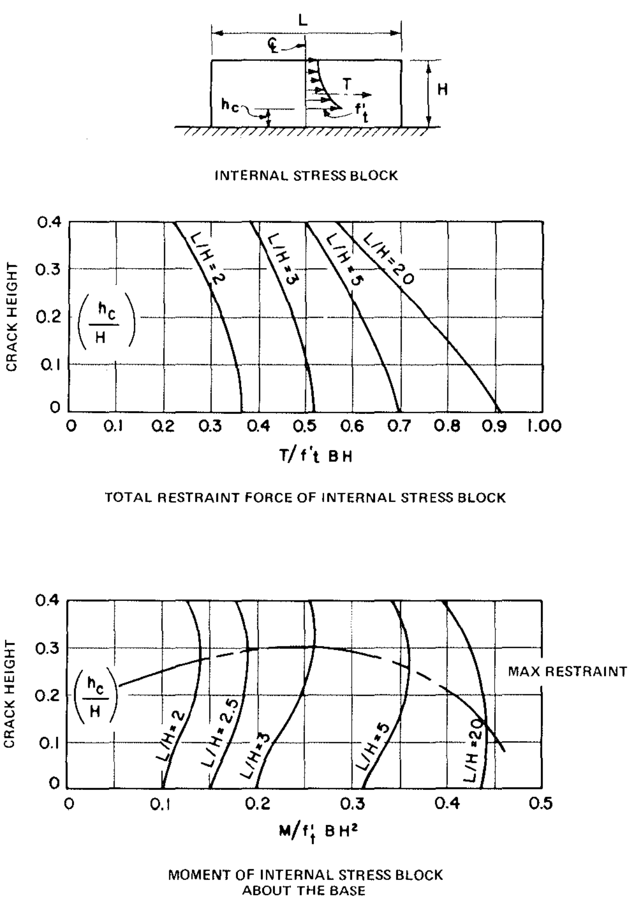


Fig. 2.3—Effect of crack propagation on internal forces

initiation of cracking can be determined from the internal stress block for various L/H ratios, and is shown in Fig. 2.2. Since cracks do not immediately propagate to the full block height throughout the member a driving force of continuing volume change must be present. If we assume that the stress at the top of a propagating crack is equal to the tensile strength of the concrete and that the increase in tensile stresses at points above the crack is proportional to the increase in stress at the top of the crack, then Fig. 2.3 shows the effect of crack propagation on restraint force and moment. From Fig. 2.3, a crack propagates with a decreasing stress field or force as previously discussed; however, the restraining moment must increase until the crack generally reaches from 0.2 to 0.3 of the height of section before the crack can propagate at a lower restraining moment. From Fig. 2.3 the maximum restraint at the base centerline of a block having an L/H ratio of 2.5 is approximately $0.2 f_t' BH^2$. This then may be assumed as the minimum base restraint which is capable of producing full block cracking for a minimum full block crack spacing of approximately $1.25H$ in unreinforced members.

In reinforced concrete walls or slabs the reinforcement becomes stressed as crack propagation reaches the steel. The effect of the stressed steel is to alter the redistribution of stress at the base which would otherwise occur without reinforcement. The moment of the steel stressed throughout the height of the crack adds directly to the restraining moment of the internal stress block at the centerline between cracks. When the combined internal stress moment and steel stress moment equals $0.2 f_t' BH$ then the combined restraint is sufficient to produce full block height cracking at the centerline between cracks.

If control of crack widths requires a minimum crack spacing less than $2H$ then reinforcement must be added to assure this spacing. From the above postulations, if the required spacing is L' then the restraining moment of the reinforcing steel at the existing crack spacing of $2L'$ would be $0.2 f_t' BH^2$ minus the restraining moment of Fig. 2.2 for $L/H = 2L'/H$.

A linear approximation of this difference can be determined by:

$$M_{RH} = 0.2 f_t' BH^2 \left(1 - \frac{L'}{2H}\right) \tag{2.2}$$

where:

M_{RH} = the restraint moment required of the reinforcing steel for full height cracking

f_t' = the tensile strength of the concrete

H = the height of block

B = the width of block

2.3—Discontinuous external or end restraint

When the contact surface of the concrete element under restraint and the supporting element is discontinuous, restraint to volume change remains concentrated at fixed locations. This is typical of all concrete elements spanning between supports. It is also typical for the central portions of members supported on material of low tensile strength or of lower shear strength than concrete requiring substantial frictional drag at the ends to develop restraint.

2.3.1 *Stress distribution of members spanning between supports*—A member which is not vertically supported throughout its length is subject to flexural stress as well as stress due to length change. When a decrease in volume or length occurs in conjunction with flexural members spanning between supports, additional rotation of the cross sections must occur. If the supports themselves are also flexural members, a deflection will occur at the top of the supports and this deflection will induce moments at the ends of the member undergoing volume change. These moment stresses will be in addition to the tensile stresses induced by the shear in the deflected supports. (See Fig. 2.4.) The end moments thus induced will increase tensile strains in the bottom face and decrease tensile strains in the top face of the member undergoing volume change. The magnitude of induced stress depends on the relative stiffnesses of the concrete element under restraint and the supporting members and may be determined when the degree of restraint K_R has been determined for the support system. For members spanning between two supports, the degree of restraint can be approximated by:

$$K_R = \frac{1}{1 + \dfrac{A_B h^3}{4LI_c}} \qquad (2.3)$$

where:

L and A_B = the length and area respectively of the member undergoing volume change

I_c and h = the average moment of inertia and height respectively of the two supporting end members

The change in bottom face steel stress for members spanning between flexural supports can be approximated by:

$$\Delta f_s = \frac{K_R C_T T_E E_S}{2pnj}\left[\frac{h}{d}\left(\frac{K_f}{K_f + K_c}\right) + 4pnj\right] \qquad (2.4)$$

where:

C_T is the linear thermal coefficient

T_E is the design temperature change including shrinkage effects

E_S is the elastic modulus of the steel

K_f is the stiffness of the beam or floor system undergoing volume change

K_c is the average stiffness of the vertical restraining elements subject to deflection by the volume change.

For complicated frames and members spanning continuously over more than two supports, the stress induced in the member from the change in volume should be determined by a frame analysis considering the effects of sidesway, member elongations under direct load, and shear deflections of the support members.

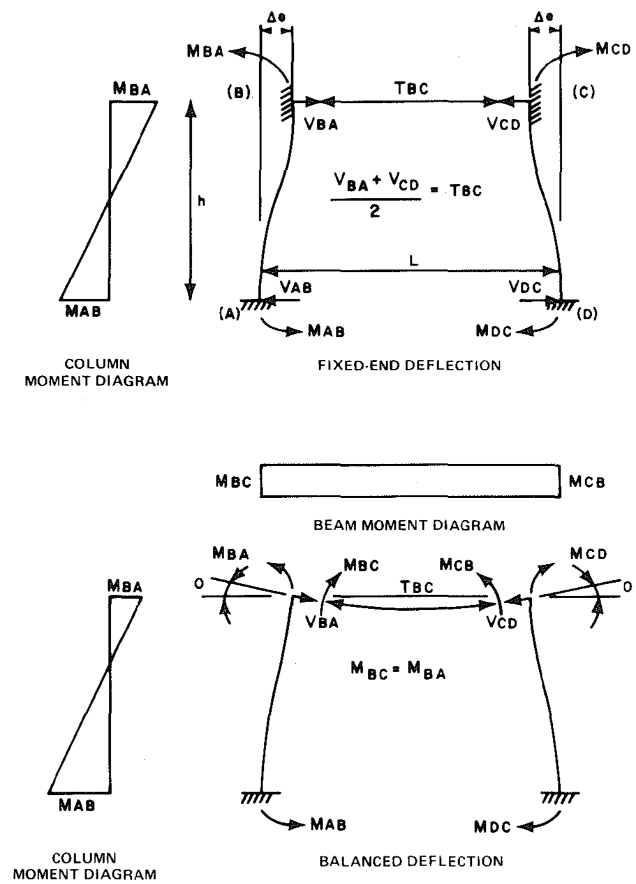


Fig. 2.4—Flexure of a simple frame induced by beam shortening

If the supporting members are very stiff relative to the member undergoing volume change, the deflection at the top of the supporting members will be essentially a shear deflection and no end moments will be induced in the member. Under these conditions the change in steel stress throughout the member will simply be:

$$\Delta f_s = 2K_R C_T T_E E_S \qquad (2.5)$$

A temperature gradient through a wall or slab with ends fixed or restrained against rotation will induce bending stresses throughout the member. When the restraint to rotation is sufficient to crack the member, cracking will be uniformly spaced throughout. Rotational stiffness is dependent on the moment of inertia of the cracked section. The ratio of the moments of inertia of cracked to uncracked sections in pure bending is $6jk^2$. Using this, the fixed-end moment for a cracked section would be:

$$\text{FEM} = (T_1 - T_2)\, C_T E_c b d^2 \left(\frac{jk^2}{2}\right) \qquad (2.6)$$

where:

$T_1 - T_2$ is the temperature difference across the member

C_T = the expansion coefficient of the concrete

2.3.2 *Stress distribution of vertically supported members*—The distribution of stresses due to volume change in members subject to a discontinuous shear restraint at the base, but vertically supported throughout its length, is dependent on the L/H ratio of the member and for all practical purposes is the same as Fig. 2.1 where L is the distance between points of effective shear transfer at the base. As the L/H ratio approaches infinity, the distribution of stress approaches uniformity over the cross sectional area at any appreciable distance from the support. (See Fig. 155 of Reference 2.)

For slabs placed on subgrade material of little or no tensile strength and lower shear strength than concrete, the distance between points of effective shear transfer depends on the frictional drag of the slab ends. A decrease in slab volume will curl the ends of the slab upward. Cracking will initiate at approximately the center of the base when the full depth of the member has a parabolic tensile stress distribution with the stress at the base equal to the tensile strength of the concrete. The cracking moment for this internal stress distribution will be $f_t'BH^2/10$. The balancing external restraining moment depends entirely on the weight of the concrete and the distribution of the base pressure. For a parabolic base pressure distribution, for the curling slab, the restraining moment will equal $0.075WBHL^2$, or

$$\frac{f_t'BH^2}{10} = 0.075\ WBHL^2$$

For $f_t' = 300$ psi, and $w = 144$ pcf.

$$L = 20\sqrt{H} \text{ (For } L \text{ and } H \text{ in ft)}$$

When the overall slab length exceeds $20\sqrt{H}$, the distribution of stress in the central portion of the slab will approximately equal that of continuously restrained base having an L/H ratio of $(L - 20\sqrt{H})/H$. When the spacing of cracks must be less than $20\sqrt{H}$, reinforcement must be provided. When the ratio of $(L - 20\sqrt{H})/H$ is less than 2, a minimum tensile force of $f_t'BH/3$ must be provided by the reinforcing steel to provide multiple cracks between the end sections. If the ratio of $(L - 20\sqrt{H})/H$ is greater than 2.5 the reinforcement must be capable of developing the full drag force of the end sections. This would be the full tensile force "T" of Fig. 2.2 for L/H corresponding to $(L - 20\sqrt{H})/H$. Thus the reinforcement requirements are:

$$A_s = \frac{T}{f_s} \geqq \frac{f_t'BH}{3f_s} \tag{2.7}$$

where:

f_t' = the tensile strength of concrete

f_s = the allowable steel stress

2.3.3 *Cracking pattern of vertically supported members*—When the stress of a member subject to discontinuous restraint or restrained at its ends exceeds the tensile strength of the concrete, a single crack will form between the points of restraint. Any additional cracking of the member must be provided by enough reinforcing steel at a controlled stress level to equal the total restraint force induced at the member ends.

2.4—Internal restraint

Internal restraint exists in members with nonuniform volume change on a cross section. This occurs, for example, within walls, slabs, or masses with interior temperatures greater than surface temperatures. It also occurs in slabs projecting through the walls of buildings with cold outside edges and warm interiors and in walls with the base or lower portions covered and the upper portions exposed to the air.

Internal restraint depends on the differential volume change within a member. Its effects add algebraically to the effects of external restraint except that their summation will never exceed

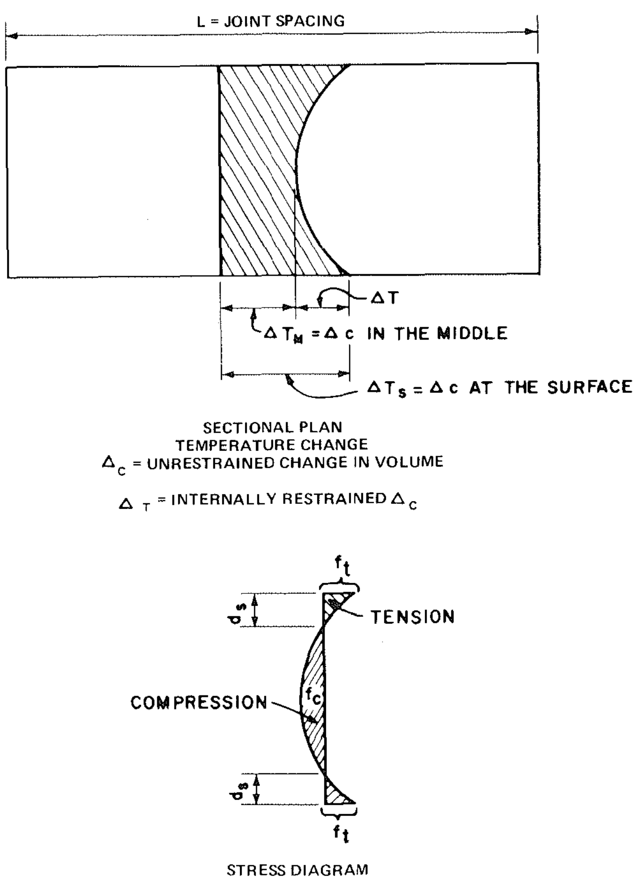


Fig. 2.5—Internal restraint

the effects of 100 percent external restraint. Therefore, where high external restraint conditions exist the effects of internal restraint may be negligible.

2.4.1 *Stress distribution and cracking*—Internal restraint is similar to continuous edge restraint, except that the effective restraining plane is the plane of zero stress in the internal stress block and is dependent on the actual temperature gradient in the concrete. (See Fig. 2.5.) For section stability the summation of tensile stress induced by the temperature gradient on a cross section must be balanced by an equal compressive force. This balance line locates the depth (d_s) of internal stress block. If the depth of the tensile stress block (d_s) is large in comparison to the spacing of joints (L), then the stress induced by volume change will not be significant. As an example, the annual temperature cycle for a 100 ft thick dam would have a 15 ft deep tensile stress block using the distribution shown in Fig. 5.3.5 of Reference 1. If we assume a 50 ft spacing of joints, the L/ds ratio would be 3.3 and the degree of restraint at the surface would be 0.25 percent using Fig. 2.1 of this report and L/ds as L/H. In contrast from the same chart the daily cycle shows a penetration of only 2 to 2.5 ft. Using 2 ft as d_s, the degree of restraint at the surface would be approximately 85 percent and assuming a concrete tensile strength of 300 psi, a concrete modulus of 3×10^6 psi and a coefficient of thermal expansion of 5×10^{-6} in. per in. per F, cracking would occur at the face with a 24 F drop in surface temperature. For equal stress the annual temperature variation would have to be 82 F. Cracking from the daily temperature cycle is not usually significant in dams and large masses, particularly in moderate climates, because of the limited penetration or significance of such cracks. The 24 F drop in mean daily temperature corresponds to normal winter temperature fluctuations for moderate climates. See Chapter 5 of Reference 1 for a more complete discussion of surface cracking.

Temperatures through a wall or slab may be nonuniform because of a difference in exposure conditions on opposite faces. Temperature distribution of this sort will curl the slab or wall if unrestrained, or induce bending stresses along the member if its ends are restrained as previously discussed in Section 2.3.1.

The plane of zero stress of the tensile stress block for projecting portions of concrete walls or slabs may be determined by a heat flow analysis or by trial as described above. The portion of cold volume to total volume is larger for members of this type than for dams or other large concrete masses. The penetration of the daily temperature cycle may therefore be assumed somewhat more than the 2 to 2.5 ft penetration previously mentioned for dams. Restraint at the free edge may also be determined for these cases from Fig. 2.1 by setting the depth of the tensile stress block (d_s) as a fixed plane 3 ft inside the exterior surface or by the following:

$$K_R = \frac{1}{\dfrac{1 + 2d_s}{W - 2d_s}} \qquad (2.8)$$

where:

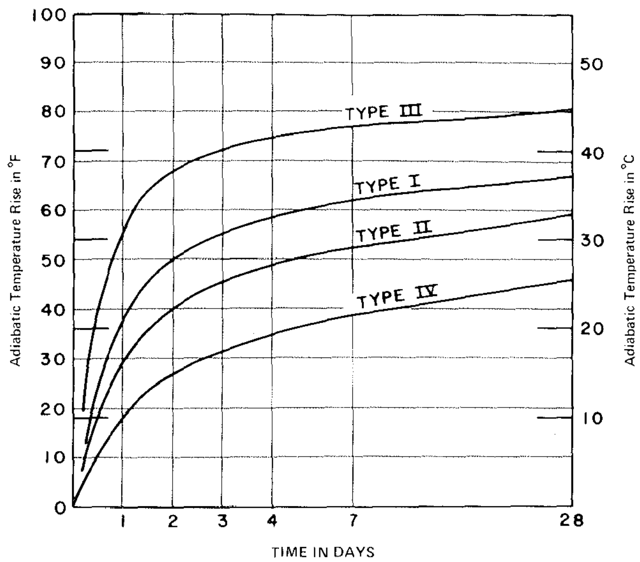
W = the total width of slab or overall height of wall

CHAPTER 3—VOLUME CHANGE

The behavior of mass concrete has been thoroughly discussed in Chapter 5 of Reference 1 and various other references contained in the Committee 207 report, "Mass Concrete for Dams and Other Massive Structures." The purpose of this chapter is to offer some practical guidance in the magnitude of volume change which can be expected in reinforced concrete structures or elements. Such structures are apt to utilize cement of higher heat generation, smaller aggregate, wetter concrete, and less temperature control than normally used or recommended for mass concrete.

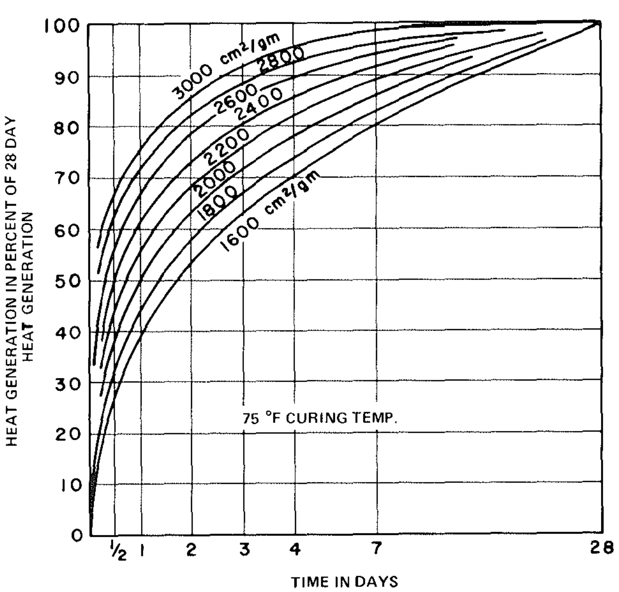
In reinforced concrete elements the primary concern is with the volume changes resulting from thermal and moisture changes. Autogenous changes, which are usually insignificant, and unpredictable actions such as alkali aggregate expansion are not considered. Volume changes due to expansive cements are also not considered.

The change in temperature to be considered in the design of reinforced concrete elements is the difference between the peak temperature of the concrete attained during early hydration (normally within the first week following placement) and the minimum temperature to which the element will be subjected under service conditions. The initial hydration temperature rise produces no stress in the concrete in changing from a plastic state to solid concrete, and the modulus of elasticity of young concrete is so small that compressive stresses induced by the rise in temperature are insignificant even in zones of restraint and can be ignored.



Cement Type	Fineness ASTM C 115 cm^2/gm	28-Day Heat of Hydration Calories per gm
I	1790	87
II	1890	76
III	2030	105
IV	1910	60

Fig. 3.1—Temperature rise of mass concrete containing 376 pcy (223 kg/m³) of cement



*ASTM C 115

Fig. 3.2—Rate of heat generation as effected by fineness of cement* for cement paste curing at 75 F (24 C)

3.1—Chemical reactions and heat generation

The rate and magnitude of heat generation of the concrete depends on the amount per unit volume of cement and pozzolan (if any), the compound composition and fineness of cement, and upon the temperature during hydration of

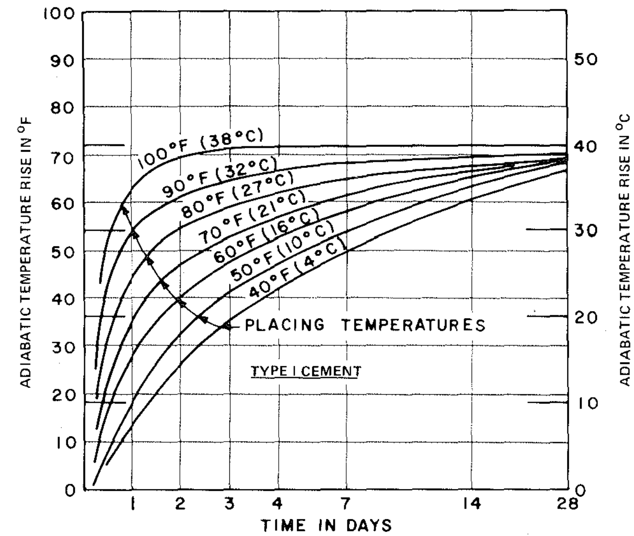


Fig. 3.3—Effect of placing temperature on temperature rise of mass concrete containing 376 pcy (223 kg/m³) of Type I cement

the cement. The hydration temperature is affected in turn by the amount of heat lost or gained as governed by the size and exposure conditions of the member. Thus, it can be seen that the exact temperature of the concrete at any given time depends on many variables.

In Fig. 3.1 are shown curves for adiabatic temperature rise versus time for mass concrete placed at 73 F and containing 376 lb/cu yd for various types of cement. These curves are typical of cements produced prior to 1960. The same cement types today may vary widely from those because of increased fineness and strengths. Current specifications only limit the heat of hydration directly of Type IV cements or of Type II cements if the purchaser specifically requests heat of hydration tests. Heat of hydration tests present a fairly accurate picture of the total heat generating characteristics of cement at 28 days because of the relative insensitivity with age of the total heat generating capacity of cement at temperatures above 70 F. At early ages, however, cement is highly sensitive to temperature and therefore heat of solution tests which are performed under relatively constant temperatures do not reflect the early age adiabatic temperature rise.

The fineness of cement also effects the rate of heat generation more than it effects the total heat generation in much the same fashion as temperature. The rate of heat generation as effected by cement fineness and placing temperature is shown in Fig. 3.2 and 3.3 respectively. These two figures are based on extrapolation of data from a study of the heats of hydration of cements by Verbeck and Foster.[3]

There are no maximum limitations on cement fineness in current specifications. By varying both fineness and chemical composition of the various types of cement, it is possible to vary widely the rate and total adiabatic temperature rise of the typical types shown in Fig. 3.1. It is therefore essential that both the fineness and chemical composition of the cement in question be considered in estimating the temperature rise of massive concrete members.

For a given fineness the chemical composition of cement has a relatively constant effect on the generation of heat beyond the first 24 hr. This can be shown using Fig. 3.2 for effect of fineness and assuming the difference in 28 day adiabatic temperatures as constant after the first 24 hr to compare the adiabatic curves of the different types of cement of Fig. 3.1 during the first week. The 28 day adiabatic temperature rise in degrees F may be calculated by:

$$Ha = \frac{1.8\,(\text{Cal/gm})\,(\text{lb of cement})}{0.22\,(150)\,(27)} \text{ in F} \quad (3.1)$$

where:

0.22 and 150 is the specific heat and density respectively of the concrete and (Cal/gm) is the 28-day measured heat generation of the cement by heat of hydration as per ASTM C 186. For a concrete mix containing 376 lb of cement per cu yd: $Ha = 0.76$ (Cal/gm) in F.

The total quantity of heat generated at any age is directly proportional to the quantity of cement in the concrete mix. When fly ash or other pozzolans are used the total quantity of heat generated is directly proportional to an equivalent cement content (C_{eq}) which is the total quantity of cement plus a proportion of total pozzolan content. The contribution of pozzolans to heat generation as equivalent cement varies with age of concrete, type of pozzolan, the fineness of the pozzolan compared to the cement, and heat generating characteristics of the cement and pozzolan itself. It is best determined by testing the combined portions of pozzolan and cement for fineness and heat of hydration and treating the blend in the same fashion as a type of cement. In general the relative contribution of the pozzolan as equivalent cement increases with age of concrete, fineness of pozzolan compared to cement, and with lower heat generating cements. Fly ash is generally lower in fineness, water requirements, and heat contribution than other pozzolans. The early age heat contribution of fly ash may conservatively be estimated to range between 15 and 35 percent as equivalent cement while other pozzolans may contribute from 5 to 10 percent more depending on the pozzolan. In general the low percentages correspond to combined finenesses of fly ash and cement as low as two-third to three-fourth of the cement alone while the higher percentages correspond to finenesses equal to or greater than the cement alone.

The rate of heat generation as affected by initial temperature, member size, and environment is difficult to assess because of the complex variables involved. The problem is somewhat simplified if we assume that the placing temperature and ambient air temperature are the same. We can then make a correction for the actual differences considering the size or volume to surface ratio of the member in question. Peak concrete temperatures for reinforced concrete structures may occur any time during the first week depending on member size, type of cement, and concrete placing temperature. Fig. 3.4 shows the effect of placing temperature and member size on the age at which peak concrete temperatures occur for concrete containing Type 1 cement. Time would be shortened or lengthened for cements of higher or lower heat-generating characteristics. For comparative purposes the early age heat generation of a Type 3 cement is approximately equivalent to a Type 1 cement at a 20 F higher placing temperature. In similar fashion the heat-generating characteristic of Types 2 and 4 cement correspond closely to that of Type 1 cement at 10 and 20 F lower placing temperatures, respectively. Fig. 3.4 shows that a large range of concrete member sizes and placing conditions will peak in 15 to 18 hr time. The approximate maximum temperature rise for concrete members containing four bags (376 lb) of Type 1 cement for placing temperatures ranging from 50 to 100 F are given in Fig. 3.5 assuming ambient air temperatures equal to placing temperatures. Corrections are required for different types and quantities of cementitious materials. A correction

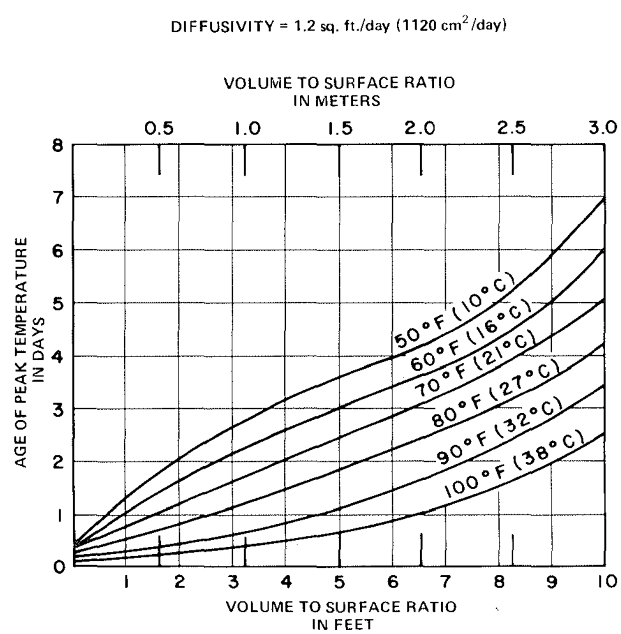


Fig. 3.4—Effect of placing temperature and exposure on age of peak temperature. Type I cement. Air temperature equals placing temperature

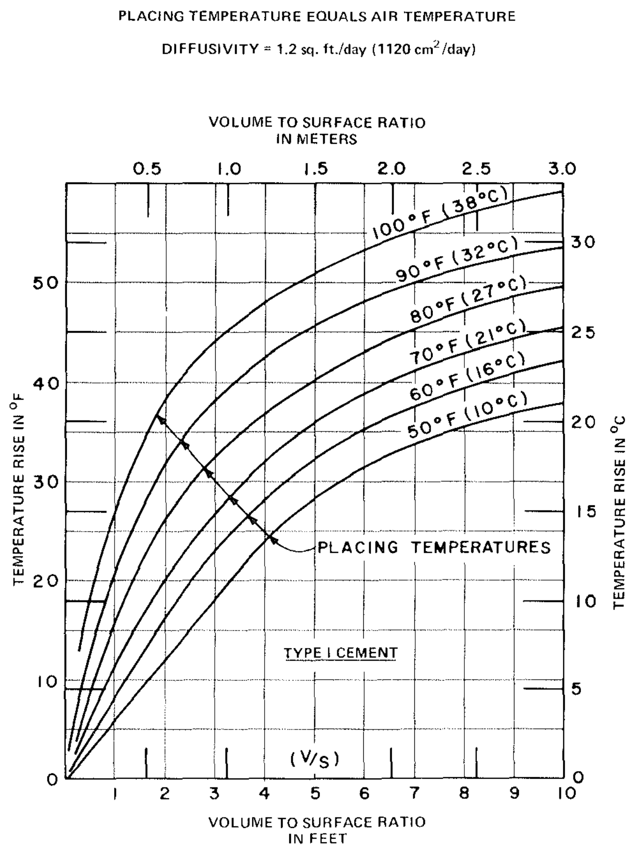


Fig. 3.5—Temperature rise of concrete members containing 376 pcy (223 kg/m³) of cement

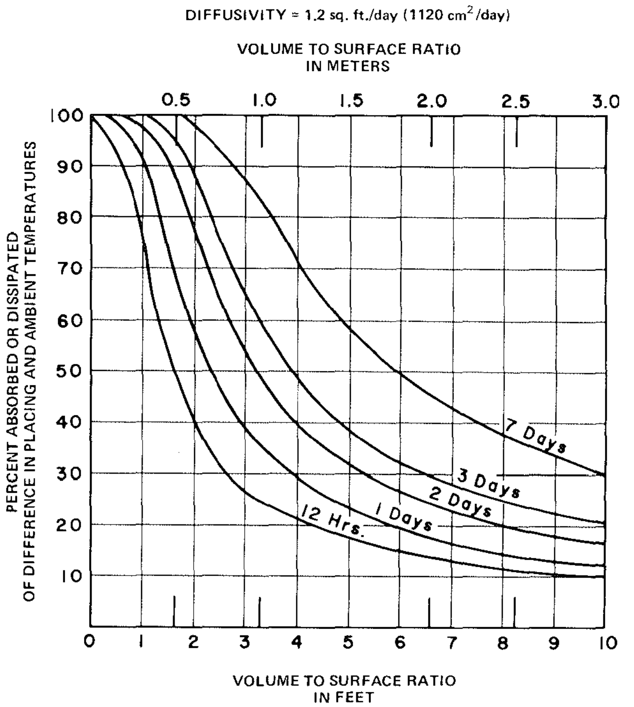


Fig. 3.6—Heat flow between air and concrete for difference in placing and ambient air temperatures

for the difference in air and placing temperatures can be made using Fig. 3.6 by estimating the time of peak temperature from Fig. 3.4. The effect of water-reducing, set-retarding agents on the temperature rise of concrete is usually confined to the first 12 to 16 hr after mixing, during which time these agents have the greatest effect on the chemical reaction. Their presence does not alter appreciably the total heat generated in the concrete after the first 24 hr and no corrections are herein applied for the use of these agents.

3.2—Moisture contents

The volume change resulting from drying shrinkage, for tensile stress considerations, is similar to volume change from temperature except that the loss of moisture from hardened concrete is extremely slow compared with the loss of heat. Drying shrinkage therefore depends on the length of drying path and often affects only the concrete near a surface. When the drying path or volume to surface ratio is small, drying shrinkage adds to the stresses induced by external restraint and should be considered in the design of the reinforcement. When the volume to surface ratio is large, the restraint to drying shrinkage is entirely internal and the result is tension on the surface or an extensive pattern of surface cracks extending only a short distance into the concrete. Surface cracks of this nature when they do occur are small and reinforcement is not particularly effective in altering the size or spacing of these cracks. Reinforcement is also not the answer for cracks resulting from plastic shrinkage.

A 24 in. thick slab will lose approximately 30 percent of its evaporable water in 24 months of continuous exposure with both faces exposed to 50 percent relative humidity.[4] If we assume a total drying shrinkage potential at the exposed faces of 300 millionths, then the average drying shrinkage for a 24 in. slab under this exposure would be 90 millionths. Concrete is not usually exposed to drying conditions this severe. From Fig. 7 of the USBR Concrete Manual,[5] 300 millionths drying shrinkage would correspond to a unit water content of the fresh concrete of 225 lb. Drying shrinkage will increase, according to the USBR chart, at the rate of 300 millionths for each 100 lb increase in water content of the mix; therefore, the drying shrinkage for the above described slab would double if the mix contained as much as 325 lb of water. Drying shrinkage is also effected by the type of aggregate used. "In general, concretes low in shrinkage often contain quartz, limestone, dolomite, granite, or feldspar, whereas those high in shrinkage often contain sandstone, slate, basalt, trap rock, or other aggregates which shrink considerably of themselves or have low rigidity to the compressive stresses developed by the shrinkage of paste."[6] In this discussion an aggregate low in shrinkage qualities is assumed. Drying shrinkage may vary widely from the values used herein depending on the aggregate.

In the design of reinforcement for exterior restraint to volume change, it is more convenient to design only for temperature change rather than for temperature and shrinkage volume change; therefore, it is desirable to express drying shrinkage in terms of an equivalent change in concrete temperature. Creep can be expected to significantly reduce the stresses induced by drying shrinkage because of the long period required for full drying shrinkage to develop. We have therefore assumed a potential drying shrinkage of 150 millionths and an expansion coefficient of 5×10^{-6} as a basis in establishing the following formula for equivalent temperature drop. We also assume that when the drying path (V/S) is more than 15 in. shrinkage will not effect external restraint calculations.

$$T_{DS} = \left(30 - \frac{2V}{S}\right)\left(\frac{W_u - 125}{100}\right) \qquad (3.2)$$

where:

W_u = the unit water content of the fresh concrete but not less than 225 lbs

V = the total volume in cu in.

S = the area of the exposed surface in sq in.

3.3—Temperature

In many structures the most important temperature considerations are the average air temperatures during and immediately following the placement of concrete, and the minimum average temperature in the concrete which can be expected during the life of the structure. The temperature rise due to hydration may be small, particularly in thin exposed members, regardless of the type or amount of cement used in the mix if placing and cooling conditions are right. On the other hand, the same member could have a high temperature rise if placed at high temperature in insulated forms.

Specifications usually limit the maximum and minimum placing temperatures of concrete. "Recommended Practice for Hot Weather Concreting" by ACI Committee 605[7] recommends a maximum placing temperature of 90 F, but suggests placing temperature of 60 F as desirable. The placing temperature of concrete during hot weather may exceed the mean daily ambient air temperature from 5 to 10 F unless measures are taken to cool the aggregate or the concrete. Corrections should be made for the difference in air temperature and placing temperature using Fig. 3.6. The maximum placing temperature in summer should be the high average summer temperature for a given locality but not more than 90 F.

Minimum temperature requirements for the first 72 hr after placing concrete are given in line 7 of Table 1.4.1 of "Recommended Practice for Cold Weather Concreting" by ACI Committee 306.[8] These minimums establish the lowest placing temperatures for consideration. Placing temperatures for spring and fall can reasonably be considered to be about half way between the summer and winter placing temperatures.

The minimum expected final temperatures of concrete elements are as varied as their prolonged exposure conditions. We are primarily concerned with the final or operating exposure conditions since cracks which may form or open during colder construction conditions may be expected to close during operating conditions provided steel stresses remain in the elastic range during construction conditions. Heated interiors would rarely be heated to temperatures less than 60 F and closed unheated interiors would rarely fall below 50 F. Minimum concrete temperatures can be conservatively taken as the average minimum exposure temperature occurring during a period of approximately one week. The mass temperature of earth or rock against concrete walls or slabs forms a heat source which affects the average temperature of concrete members depending upon the cooling path or volume to surface ratio of the concrete. This heat source can be assumed to effect a constant temperature T_M at some point 8 to 10 ft from the exposed concrete face.

The minimum temperature of concrete against earth or rock mass can be approximated by:

$$T_{min} = T_A + \frac{2(T_M - T_A)}{3}\sqrt{\frac{V/S}{96}} \qquad (3.3)$$

where:

T_A = the minimum expected average ambient air temperature in deg F over a prolonged exposure period of approximately one week.

T_M would vary from approximately 40 to 60 F (4 to 16 C) depending on climate.

V/S = the volume to exposed surface ratio in in.

3.4—Heat dissipation and cooling

Means of determining the dissipation of heat from bodies of mass concrete are discussed in Reference 1 and can readily be applied to massive reinforced structures. Reinforced elements or structures do not generally require the same degree of accuracy in determining peak temperatures as unreinforced mass concrete. In unreinforced mass concrete peak temperatures are determined for the purpose of preventing cracking. In reinforced concrete cracking is presumed to occur and the consequences of overestimating or underestimating the net temperature rise is usually minor compared to the overall volume change consideration. Sufficient accuracy is normally obtained by use of charts or graphs such as Fig. 3.5 to quickly estimate the net temperature rise for concrete members cooling in a constant temperature environment equal to the placing temperature, and by use of Fig. 3.6 to account for the difference in the actual and assumed cooling environment.

Fig. 3.5 gives the maximum temperature rise for concrete containing 376 lb of Type I portland cement per cu yd of concrete in terms of volume to surface ratio of the member. Volume to surface ratio actually represents the average path length through which heat is dissipated from the concrete. The average path length will always be less than the minimum distance between faces. In determining the volume to surface ratio consider only the surface area exposed to air or cast against forms. Steel forms, without insulation can be ignored; however, wood forms or steel forms with insulation must be considered in the minimum flow direction on the basis of the effective increase in flow length due to the insulating effect. Each in. of wood has an equivalent insulating value of about 20 in. of concrete but can, for convenience, be assumed equivalent to 2 ft of additional concrete. Any faces farther apart than 20 times the thickness of the member can be ignored as contributing to heat flow. Volume to surface ratio can best be determined by multiplying the calculated volume to surface ratio of the member, excluding the insulating effect of forms by the ratio of the minimum flow path including forms divided by the minimum flow path excluding forms. For slabs, the V/S ratio should not exceed three fourths of the slab thickness. While multiple lift slabs are not generally classed as reinforced slabs, the V/S ratio should not exceed the height of lift if ample time is provided for cooling lifts.

The temperature rise for other types of cement and for mixes containing differing quantities of cement or cement plus pozzolan from 376 lb can be proportioned as per Section 3.1 above.

Fig. 3.6 accounts for the difference in placing temperatures and ambient air temperatures. Volume to surface ratios for Fig. 3.6 should be identical to those used with Fig. 3.5. In all previous temperature determinations the placing temperature has been assumed equal to ambient air temperature. This is often not the case because of the cooling measures required for the concrete during hot weather and the heating measures required during cold weather. When the placing temperature of concrete is lower than the average ambient air temperature, heat will be absorbed by the concrete such that only a proportion of the original temperature difference will be effective in lowering the peak temperature of the concrete. When the placing temperature is higher, the opposite effect is obtained. As an example assume for an ambient air temperature of 75 F that the placing temperature of a 4 ft thick wall 12 ft high is 60 F instead of 75 F. The (V/S) ratio would be 3.4 ft assuming wooden forms. The age for peak temperature would be 2.3 days from Fig. 3.4. From Fig. 3.6, 50 percent of the heat difference will be absorbed or 7.5 F; therefore, the base temperature or the effective placing temperature for determining temperature rise will be 68 F. If no cooling methods are used the actual placing temperature of the concrete will be 85 F, the age of peak temperature would be 1 day, and the base temperature or effective placing temperature for determining temperature rise will be 81 F.

3.5—Summary and examples

The maximum effective temperature change constitutes the summation of three basic temperature determinations. They are: (1) the difference between effective placing temperature and the temperature of final or operating exposure conditions, (2) the temperature rise of the concrete due to hydration, and (3) the equivalent temperature change to compensate for drying shrinkage. Measures for making these determinations were previously discussed, therefore, the following example problems employ most of the calculations required in determining the maximum effective temperature change.

Example 3.1—A 2 ft wide retaining wall with rock base and backfill on one side; 20 ft high, and 100 ft long placed in two 10 ft lifts, wood forms; summer placing with concrete cooled to 60 F; concrete mix designed for a specified strength of 3000 psi or average strength of 3700 psi at 90 days contains 215 lb of Type II cement (adiabatic curve same as Fig. 3.1), 225 lb of fly ash and 235 lbs of water per cu yd.

1. Determine the volume to surface ratio

$$V/S = \left(\frac{2\,(10)}{2\,(10)+2}\right)\frac{2+4}{2} = 2.7 \text{ ft}$$

2. Determine the difference between effective placing temperature and final exposure temperature:

   (a) Establish ambient air temperature for summer placement based on locality. Assume 75 F average temperature

   (b) Concrete peaks at 2 days from Fig. 3.4. Using Fig. 3.6 the heat absorbed for a $V/S = 2.7$ is approximately 60 percent

   (c) Net effective placing temperature $T_{pE} = 60 + 0.6\,(15) = 69$ F

   (d) Establish minimum exposure temperature for 1-week duration. Assume 20 F

(e) For final exposure conditions V/S equals approximately 24 in. since heat flow is restricted to one direction by the backfill. For two faces exposed V/S would equal approximately 12 in.

(f) $T_{min} = 20\text{ F} + 2/3\ (60\text{-}20)\ \sqrt{24/96} = 33.5\text{ F}$, say 34 F

(g) Difference $= 69 - 34\text{ F} = 35\text{ F}$

3. Determine the temperature rise:

(a) From Fig. 3.5 the temperature rise for an effective placing temperature of 69 F and V/S of $2.7 = 25\text{ F}$

(b) Correction for Type II cement peaking at

$$2\text{ days} = T_c = \frac{40}{50}\ (25) = 20\text{ F}$$

(c) Correction for mix. $C_{eq} = 215 + \frac{225}{4} = 272$

$$\text{lb } T_{C+F} = 20\text{ F}\frac{(272)}{(376)} = 14.5\text{ F, say } 15\text{ F}$$

4. Determine the equivalent temperature for drying shrinkage. Since V/S for final exposure conditions is greater than 15 in. no additional temperature considerations are required for external restraint considerations

5. The maximum effective temperature change

$$T_E = 35 + 15\text{ F} = 50\text{ F}$$

Example 3.2—Same wall as Example 1 except that no cooling measures were taken and the concrete mix contains 5 bags of a Type 1 cement having a turbidimeter fineness of 2000 cm²/gm and 28-day heat of solution of 94 cal/gm.

1. (a) With no cooling measures the placing temperature could be as much as 10 F above the ambient temperature of 75 F or $T_p = 85\text{ F}$

(b) Concrete peaks at 18 hr ±
Heat dissipated $= 0.36\ (10) = 4\text{ F}$
∴ Heat absorbed $= 6\text{ F}$

(c) Effective placing temperature $T_{pE} = 75 + 6 = 81\text{ F}$

(d) Minimum $= 34\text{ F}$

(e) Difference $= 81 - 34 = 47\text{ F}$

2. (a) Temperature rise from Fig. 3.5 for $V/S = 2.7$ and effective placing temperature of $86\text{ F} = 34\text{ F}$

(b) Correction for cement
From Fig. 3.2, difference in fineness of 2000 V/S 1800 cm²/gm @ 1½ days $= 58/52 = 1.12$
Temperature difference for heat of solution $= 0.76\,(94 - 87) = 5\text{ F}$
H_a @ 1½ days $= 1.12\ (45 + 5) = 56\text{ F}$

$$\therefore \text{Correction} = \frac{56}{45}\ (34) = 42\text{ F}$$

(c) Correction for cement content $= \frac{5\,(42)}{4} = 52\text{ F}$

3. No addition for drying shrinkage

4. The maximum effective temperature change

$$T_F = 47 + 52 = 99\text{ F}$$

In comparing the above two examples the cooling measures accounted for a net temperature difference of 20 F and the differences in concrete mixes accounted for a net difference of 29 F for a combined difference of 49 F which constitutes a 98 percent increase in volume change for Example 2 over Example 1 for the same retaining wall.

CHAPTER 4—PROPERTIES

4.1—General

This chapter discusses the principle properties of massive concrete which affect the control of cracking and provides guidance to evaluate those properties.

4.2—Strength requirements

The dimensions of normal structural concrete are usually determined by structural requirements utilizing 28-day strength concrete of 3000 psi or more. When these dimensions are based on concrete stresses in the elastic range which approach ACI 318 Code limitations, the spacing of cracks will be primarily influenced by flexure, and the resultant steel stresses induced by volume change will normally be small in comparison with flexural stresses. Under these conditions volume control measures do not have the significance that they have when concrete stresses in the elastic range are low and crack spacing is controlled primarily by volume change.

The dimensions of massive reinforced concrete sections are often set by criteria totally unrelated to the strength of concrete. Such criteria often is based on stability requirements where weight and not strength is of primary importance; on arbitrary requirements for water tightness per ft of water pressure; on stiffness requirements for the support of large pieces of vibrating machinery where the mass itself is of primary importance; or on shielding requirements as found in nuclear power plants. Once these dimensions are established they are then investigated using an assumed concrete strength to determine the reinforcement requirements to sustain the imposed loadings. In slabs the design is almost always controlled by flexure. In walls the reinforcement re-

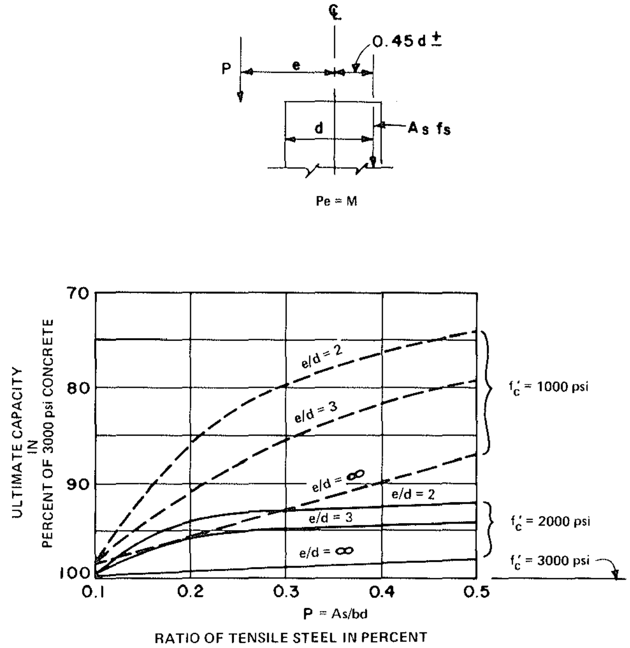


Fig. 4.1 Effect of concrete strength on ultimate capacity $f_y = 60{,}000$ psi

quirements are usually controlled by flexure or by minimum requirements as load bearing partitions. Shear rarely controls except in the case of cantilevered retaining walls or structural frames involving beams and columns.

In flexure the strength of massive reinforced sections is controlled almost entirely by the reinforcing steel. The effect of concrete strength on structural capacity is dependent on the quantity of reinforcing steel (steel ratio) and the relationship of direct load to bending moment. If the eccentricity of the loading with respect to member depth (e/d) is greater than 2, Fig. 4.1 shows the relationship of required concrete strength to structural capacity for steel ratios up to 0.005 using 3000 psi as the base for strength comparison. For steel ratios less than 0.005, there is no significant increase in structural capacity with higher strength concretes within the eccentricity limits of the chart. Most massive concrete walls and slabs will fall within the chart limits.

The principal reason for consideration of the effects of lower concrete strengths concerns the early loading of massive sections and the preeminent need in massive concrete to control the heat of hydration of the concrete. If design loading is not to take place until the concrete is 90 or 180 days old, there is no difficulty using pozzolans in designing low heat-generating concrete of 3000 psi at these ages. Such concrete will, however, have significantly lower early strengths for sustaining construction loadings. Normally, the designer only investigates those construction loads which exceed operational live loads and usually allows for a somewhat smaller factor of safety for these loads because of their temporary nature. From Fig. 4.1 it can readily be seen that for members subject to pure bending ($e/d = \infty$), less than 15 percent loss of capacity will be experienced in loading a member containing 0.5 percent steel when it has a compressive strength of only 1000 psi. It should be noted that while structural capacity is relatively unaffected by the 1000 psi strength, loading and creep deflection will be significantly larger than for 3000 psi concrete. For construction loadings this is usually not significant particularly since members with this low steel ratio have enough excess depth to offset the increase in deflection due to lower modulus of elasticity.

Most massive reinforced concrete members subjected to flexural stress will have steel ratios in the range of 0.0015 to 0.002 in the tensile face. Fig. 4.1 shows that in this range reinforced concrete in flexure is capable of sustaining up to 85 percent of the structural capacity of 3000 psi concrete with concrete strengths as low as 1000 psi. Construction loadings rarely control design and a 15 percent increase in safety factor for construction loadings will also not likely control. Therefore, for massive reinforced sections within these limits a simple restriction of limiting imposed flexural loads until the concrete achieves a minimum compressive strength of 1000 psi should be adequate.

It should be obvious from the above that for massive reinforced concrete a much higher percent of strengths less than design strength can be tolerated without incroaching on structural safety than can be allowed for normal structural concrete containing high steel ratios. From Fig. 4.1 a minimum strength of 2000 psi results is less than an 8.5 percent loss in ultimate capacity compared with 3000 psi strength.

As previously mentioned, shear strength may control the thickness of a cantilevered retaining wall. The strength of concrete in shear is approximately proportional to $\sqrt{f_c'}$ and, therefore, the loss in shear strength of a given member is greater than the loss in flexural strength for a given reduction in compressive strength. The design loading for a wall sized on the basis of shear strength is the load of the backfill and rarely will construction schedules allow the lower lifts to attain 90 to 180 days strength before completion of the backfill is needed. Since shear at the base of the wall upon completion of the backfill controls, a design based on 2000 psi will require approximately a 22 percent wider base. For tapered walls this would only mean a 11 percent increase in total volume. The 22 percent increase in base wall thickness would allow a 30 to 35 percent reduction in bending steel requirements (using USD) which would directly offset the cost of the added concrete volume, possibly resulting in a lower overall cost for the wall. By restricting the placing of backfill against any lift until it has obtained a minimum strength of 1000 psi and restricting completion of backfill until the first lift has attained 2000 psi, a reasonable schedule for backfill with respect to concrete construction can be established. A 2000 psi strength requirement at 28 days works conveniently with this type of construction requirements and will provide sufficient strength for durability under most exposure conditions particularly if 90 day strengths exceed 3000 psi.

4.3—Tensile strength

In conventional reinforced concrete design it is assumed that concrete has no tensile strength and a design compressive strength appreciably below average test strength is utilized. Neither approach is acceptable in determining reinforcing steel requirement for volume change crack control. The actual tensile strength is one of the most important considerations and it should be determined to correspond in time to the critical volume change. Since compressive strength is normally specified, it is desirable to relate tensile and compressive strength.

Both strength and potential for volume change are affected by concrete aggregates such that restrained concrete of equal w/c ratios made from crushed stone aggregates will withstand a considerably larger drop in temperature without cracking than concrete made from natural aggregates. For a given compressive strength, however, the type of aggregate does not appreciably effect tensile strength. The age at which concrete attains its compressive strength does effect the tensile-compressive strength relationship such that the older the concrete is the larger the tensile strength for a given compressive strength.

The most commonly used test to determine the tensile strength of concrete is now the splitting tensile test. This test tends to force the failure to occur within a narrow band of the specimen rather than occurring in the weakest section. If the failure does occur away from the center section, the calculations will indicate a higher than actual strength. The tensile strength for normal weight concrete is usually taken as $6.7\sqrt{f_c'}$ and drying has little effect on the relationship.

Direct tensile tests made by attaching steel base plates with epoxy resins indicate approximately 25 percent lower strengths. Such tests are significantly affected by drying.

If tensile cracks must be initiated at surfaces protected from drying, it is recommended that

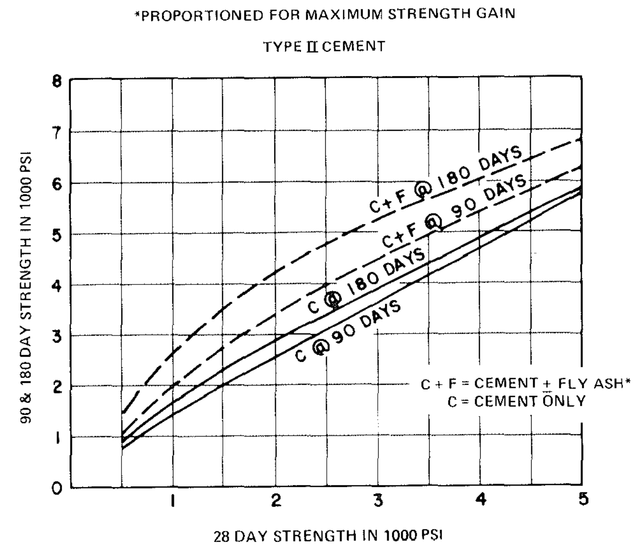


Fig. 4.2—Comparison of 28, 90, and 180 day compressive strengths

$6\sqrt{f_c'}$ be used as the tensile strength. Where surfaces are exposed to drying, $4\sqrt{f_c'}$ is recommended. An exception to the latter value is small elements where early age creep may relieve drying shrinkage stresses. In this situation, where $V/S \leqq 3$ in. the tensile strength should again be taken as $6\sqrt{f_c'}$.

In the preceeding expressions it is more appropriate to use the probable compressive strength at critical cracking rather than the specified strength. For normal structural concrete it is therefore recommended that at least 700 psi be added to the specified strength in the design of concrete mixes. For massive reinforced sections (as described in Section 4.3) it is recommended that mixes be designed for the specified strength. When strengths are specified at 28 days some additional allowance should be made for strength gain at the time of cracking particularly where fly ash is utilized. The strength gain from 28 days to 90 and 180 days age as a percent of the 28-day strength varies with the 28-day strength depending on the cement and the proportions of fly ash or other pozzolans used. For concrete mixes properly proportioned for maximum strength gain, Fig. 4.2 gives a typical comparison for mixes with and without fly ash using Type II cement. Since maximum volume change considerations are based on summer placing temperatures the above expressions relating tensile and compressive strength should be based on probable concrete strengths at 90 days to 6 months age.

4.4—Modulus of elasticity

Unless more accurate determinations are made, the elastic modulus in tension and compression may be assumed equal to $w^{1.5}\ 33\sqrt{f_c'}$ (in psi), which for normal weight concrete is $57{,}000\sqrt{f_c'}$.

It also should be based on probable strength as discussed in Section 4.3.

4.5—Creep

Creep is particularly related to elastic modulus at the time of loading and to the length of time under load. It is also affected by the age of the concrete at the time of loading and by the volume to surface ratio. Its primary effect is, as noted in Section 4.3, the relief of drying shrinkage stresses in small elements. In general, when maximum temperature changes occur over a relatively short time period, creep can only slightly modify temperature stresses.

4.6—Thermal properties of concrete

The thermal properties of concrete are coefficient of expansion, conductivity, specific heat and diffusivity. These properties have a significant effect on the change in concrete volume which may be expected and should be determined in the laboratory using job materials in advance of design, if possible. Reference 1 discusses these properties in detail and presents a broad range of measured values.

Where laboratory tests are not available, it is recommended that the thermal coefficient of expansion be assumed as 5×10^{-6} in. per in. per F for calcareous aggregate, 6×10^{-6} in. per in. per F for silicious aggregate concrete, and 7×10^{-6} in. per in. per F for quartzite aggregate.

A diffusivity of 1.2 sq ft per day has been assumed in the preparation of Fig. 3.4, 3.5, and 3.6. A concrete of higher or lower diffusivity will have the effect respectively of decreasing or increasing the volume to surface ratio, and can be accounted for by multiplying the actual V/S ratio by 1.2 divided by the measured concrete diffusivity.

CHAPTER 5—CRACK WIDTHS

5.1—General

It is desirable to limit crack widths for corrosion protection, leakage prevention or esthetic reasons. Surface crack widths are important from an esthetic viewpoint, are easy to measure, and are the subject of most limitations. Shear deformation of the concrete cover and curvature of flexural members will make a crack width at the level of the steel narrower than at the surface. The crack width at the steel is critical for corrosion. The alkalinity of concrete which inhibits the solubility of corrosion products will prevent corrosion in sound concrete at all but excessively wide cracks. Some corrosion appears to be occurring in continuously reinforced concrete pavements with maximum surface cracks approximately 0.33 in. in width where deicers are in use. Very narrow cracks just visible (0.002 in.) will probably leak at least initially; however, nonmoving cracks up to 0.005 in. may heal in the presence of excess moisture and therefore would not be expected to leak continually. Any leakage may be expected to stain the exposed concrete face. Where such staining cannot be tolerated strict volume control measures must be instituted to prevent cracking throughout the block or wall.

5.2—Limitations

Until more research is reported, arbitrary crack width limitations must be adopted. ACI 318-71[9] limits crack widths in Section 10.6 by limiting the distribution of flexural reinforcement. The 318-71 Commentary on the Building Code says that the Code limitations are based on crack widths of 0.016 in. for interior exposure and 0.013 in. for exterior exposure. Permissible crack widths versus exposure conditions in reinforced concrete are given in Table 4.1 of "Control of Cracking in Concrete Structures,"[13] by ACI Committee 224.

The principal difference between massive reinforced sections and normal reinforced concrete sections is that the ratio of distance from neutral axis to the tensile face divided by the distance from neutral axis to steel approaches 1.0 for mass concrete (particularly for nonflexural members) compared to 1.2 for normal concrete. In design of massive reinforced concrete, members of ACI Committee 207 recommend the full Gergely-Lutz expression be used accounting for this distance effect on the widths of surface cracks. For severe exposure conditions or restriction of leakage, the permissible crack widths recommended by ACI Committee 224,[13] without extensive volume control measures, will require excessively large quantities of reinforcement in most massive concrete members because of the very low steel stresses which must be utilized. The use of large size bars, closely spaced and minimum cover requirements will likely require smaller maximum aggregate sizes and wetter mixes to facilitate placement. Subsequent volume changes and cracking may therefore increase rather than decrease. For these reasons ACI Committee 207 does not recommend designing massive reinforced concrete members for crack widths less than 0.009 in. Cracks of this magnitude will allow some leakage; however, leakage will be minimum and controllable. Corrosion should also not be a concern except possibly under extreme exposure conditions. Complete watertightness requiring almost total elimination of cracking can only be achieved by volume control measures in very large members.[10,13] Crack width is subject to wide scatter. Average crack widths should be approximately two thirds of the calculated maximums[11] and some widths in excess of calculated maximums should be expected.

5.3—Calculations

A number of crack width equations are proposed in the literature. ACI 318-71 adopted an expression based on one developed in a statistical study by Gergely and Lutz reported in ACI SP-20:[12]

$$w = 0.076 \sqrt[3]{d_c A}\; R f_s\, 10^{-3} \qquad (5.1)$$

where:

w = maximum crack width at surface, in.

d_c = cover to center of bar, in.

A = average effective concrete area around a reinforcing bar ($2d_c \times$ spacing), sq in.

R = distance from neutral axis to the tensile face divided by distance from neutral axis to steel (one for tensile elements)

f_s = calculated steel stress, ksi

Although this expression was developed from flexural specimens, it appears to have reasonable correlation for tensile specimens. It should be noted that this expression limits steel stress "f_s" in direct proportion to the limitation of crack width "w."

CHAPTER 6—APPLICATION

6.1—General

The determination of restraint, volume change, appropriate concrete properties and crack widths have been discussed. They will now be combined for calculation of steel areas. Exterior loads which induce tensile stress in the concrete in addition to those induced by volume change must also be accounted for in steel area calculations.

6.2—Volume change plus flexure

The change in stress $\triangle f_s$ induced by a decrease in volume of flexural members spanning between supports (discussed in Section 2.3.1) should be added directly to the service load stress and crack width checked as per Sections 5.2 and 5.3. In lieu of crack width, ACI 318-71 checks a value of z:

$$z = f_s \sqrt[3]{d_c A} \qquad (6.1)$$

with the notation as in 318-71.

For normal structural concrete as opposed to massive concrete, the value of z should be limited to 175 for normal interior exposure, 145 for normal exterior, and 100 for severe exposure conditions. For massive reinforced concrete the combined stresses should be limited by crack width based on Chapter 5. In addition the minimum ratio of tensile steel reinforcement for massive concrete should not be less than 0.0015.

6.3—Volume change without flexure

When a decrease in volume occurs in a member not subject to flexure the limiting stress in the reinforcing steel for controlling crack widths can be determined from the crack width formula of Section 5.3 by assuming a bar cover and spacing and calculating f_s from:

$$f_s = \frac{w \times 10^3}{0.076 \sqrt[3]{d_c A}} \text{ (in ksi)} \qquad (6.2)$$

where:

w is the permissible crack width.

6.3.1 *Continuous external restraint*—Members subject to continuous restraint at their bases or on one or more edges will crack under continuing volume change as described in Section 2.2.2. Cracks are not uniform and will vary in width throughout the height of the member with the maximum crack opening occurring just above the top of the

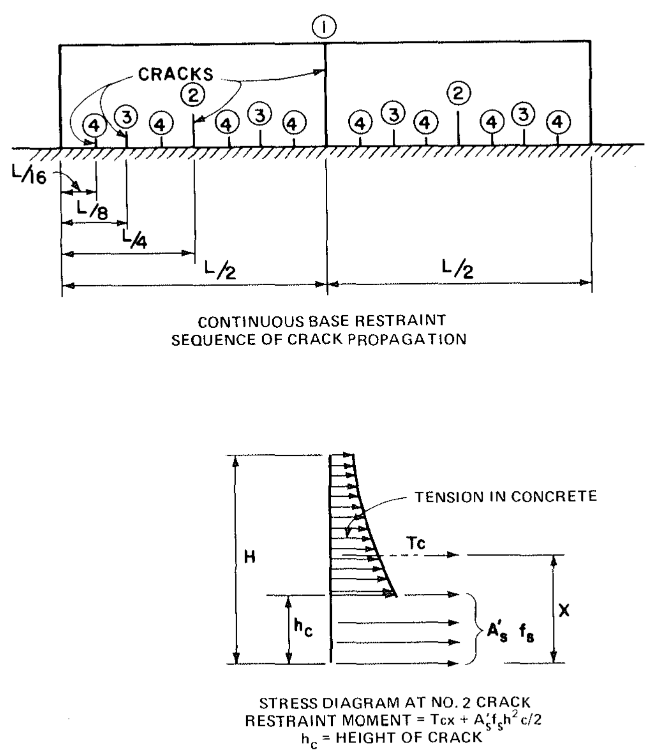


Fig. 6.1—Sequence of crack propagation and distribution of stress at No. 2 crack

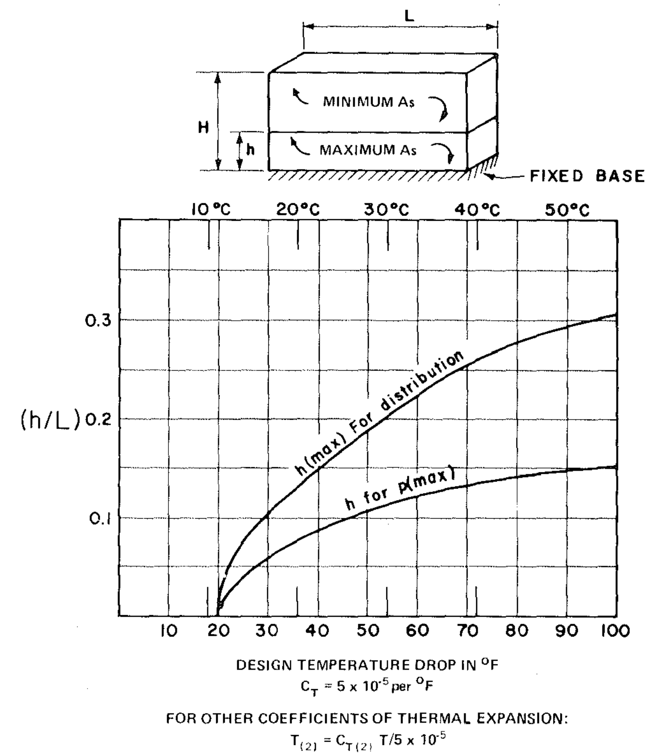


Fig. 6.2—Wall height requiring maximum temperature and shrinkage reinforcement as a ratio of base length

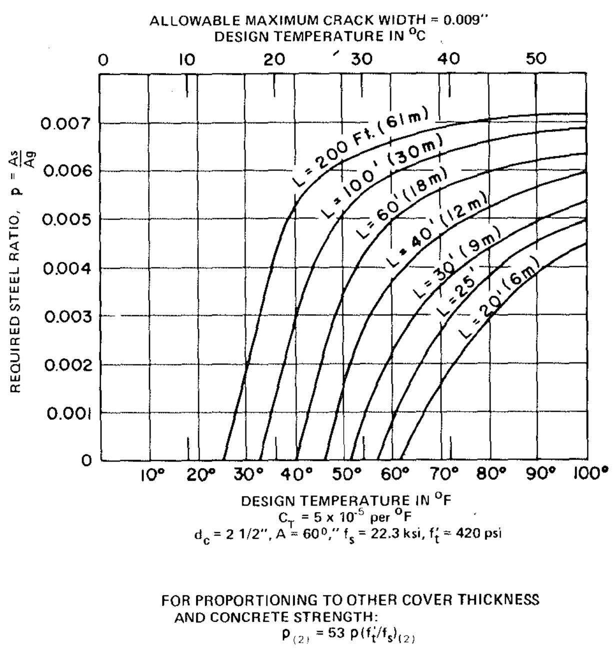


Fig. 6.3—Maximum temperature and shrinkage reinforcement walls with fixed bases

next highest extended crack assuming the cracking sequence of Fig. 6.1. Below this point there are two or more times the number of cracks to balance volume change. The concrete at the top of the partially extended crack is assumed stressed to f_t'. Therefore the summation of crack widths on any horizontal plane must approximately equal the total volume change $(K_R L C_T T_E)$ minus concrete extensibility $L\, f_t'/E_c$.

Hognestad[11] found that for the normal range of service load stress for high strength reinforcement, which is between 30 and 40 ksi, a mean value of the ratio of maximum crack width to average crack width was 1.5. If N is the number of cracks and "W" is the maximum crack width then the $NW/1.5$ will be the summation of crack widths in a given length and:

$$\frac{N_w}{1.5} = 12L\ (K_R C_T T_E - f_t'/E_c) \qquad (6.3)$$

for L in ft

If the average crack spacing equals L' then $NL' = L$ and:

$$L' = \frac{W}{18\,(K_R C_T T_E - f_T'/E_c)} \qquad (6.4)$$

It is necessary to calculate the above required average crack spacing to determine the required restraining moment to be supplied by the reinforcing steel. Cracking throughout a member may or may not extend the full height of the member, depending on the L/H relationship (see Fig. 6.1). When cracks extend only a portion of the height only the reinforcing steel below the top of the crack is effective in contributing to the internal restraint moment. (From Fig. 6.1, the internal restraint moment between full block cracks $= T_c\, x + A_s' f_s h_c^2/2$.) Even when some cracks do extend the full height others extend only part way so that the same situation applies between full height cracks. For this reason reinforcement is more effectively distributed if the wall is examined at several locations above the base to determine the average crack spacing required at each location corresponding to the degree of restraint (K_R) at each distance h from the base. The additional restraining moment $(A_s' f_s h_c^2)/2$ required of the reinforcing steel between the point h and the restrained base to produce the required crack spacing L' at h can be conservatively determined by substituting h for H in Eq. (2.2).

$$M_{Rh} = 0.20\, f_t' B h^2 \left(1 - \frac{L'}{2h}\right) \qquad (6.5)$$

The restraint factor K_R to be used in the calculation of L' at h can be read directly from Fig. 2.1 as the proportional height above the base (h/H) corresponding to the actual L/H curves. It is conservative and usually convenient to assume the distance h as the free edge distance H and read K_R in Fig. 2.1 at the free edge using L/h as L/H.

In determining the volume change reinforcement required in each face of walls with continuous base restraint, calculations at lift intervals or at some arbitrary intervals above the base should be made as follows:

$$A_b = 0.4\,\frac{f_t'}{f_s}\,\frac{B_h}{N_H}\left(1 - \frac{L'}{2h}\right) \qquad (6.6)$$

where:

N_H = the total number of bars in the h distance above the base

A_b = the size of bars required in each face of the wall. $(A_s'h/N_H) = A_b$

As the distance h from the base increases, steel requirements will first increase and then decrease. Maximum steel requirements depend on base length, effective temperature drop and coefficient of thermal expansion. Fig. 6.2 gives the point of maximum steel requirements in terms of base length and design temperature for a coefficient of thermal expansion of 5×10^{-5} in. per in. per F (9×10^{-5} cm per cm per C). The same curve can be used for other expansion coefficients by using another design temperature equal to $C_T T_E / 5 \times 10^{-5}$. Fig. 6.2 also provides the point h above which only minimum steel is required. Recommendations for minimum steel requirements are given in Section 6.4. Only minimum steel is required where L' is greater than $2h$. Fig. 6.3, 6.4, and 6.5 give the maximum steel requirements in terms of crack width, effective temperature drop, and base length for concrete walls having a $C_T = 5 \times 10^{-5}$ per F. These figures can be used to proportion steel requirements in place of the multiple calculations described above with only slightly higher total steel quantities being required. The maximum height "h" over which these steel quantities are required can be determined from Fig. 6.2. Above h, only minimum steel is required. Requirements for concrete properties and cover distances other than noted can be proportioned as shown.

For slabs with continuous base restraint or walls with one side continuously restrained:

$$A_b = \frac{0.20\,f_t'}{f_s}\left(1 - \frac{L'}{2H}\right)\frac{BH}{N_B\left(\dfrac{H - t_b}{H}\right)} \qquad (6.7)$$

where:

N_B = the total number of bars in the free face of the slab or wall.

In the case of relatively thick slabs, the amount of reinforcement required in the top face of the slab may be reduced by including the effect of the reinforcement in the sides. For this:

$$A_b = 0.20\,\frac{f_t'}{f_s}\left(1 - \frac{L'}{2H}\right)\frac{BH}{N_B\left(\dfrac{H - t_b}{H}\right) + \dfrac{N_H}{2}} \qquad (6.8)$$

Only minimum steel is required where L' is greater than $2H$ (See Section 6.4).

In applying the above formulas to relatively large masses the amount of reinforcement required will make it quite obvious that additional measures to control volume change should be used in order to control crack widths. Reinforcement is

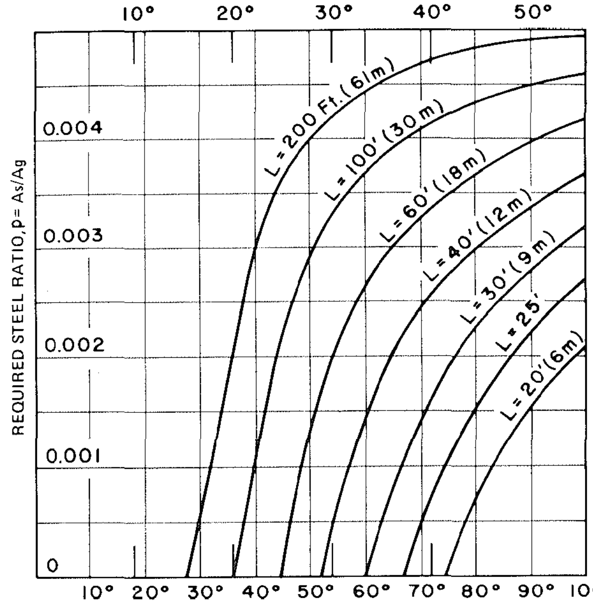


Fig. 6.4—Maximum temperature and shrinkage reinforcement walls with fixed bases

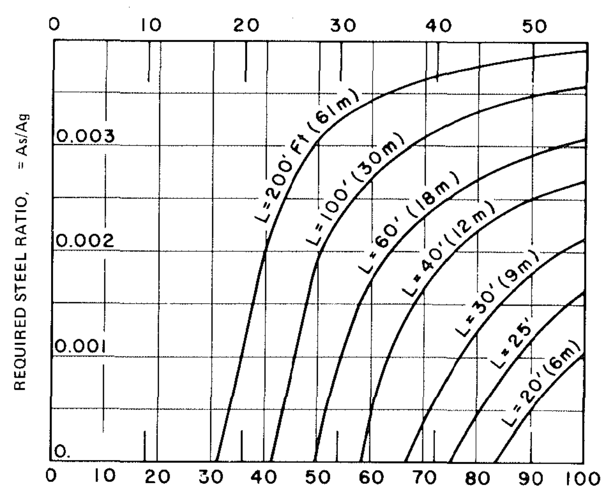


Fig. 6.5—Maximum temperature and shrinkage reinforcement walls with fixed bases

not practical in controlling the crack widths of very large externally restrained masses, and for these structures the principles of mass concrete construction described in Reference 1 must be followed to control cracking. The above formulas for crack spacing, however, can be utilized to establish a somewhat higher allowable temperature drop than normally used for mass concrete by acknowledging an acceptable crack. This can be seen in the design temperatures corresponding to zero steel requirements for the lengths of wall shown in Fig. 6.3 through 6.5.

Design temperatures in unreinforced sections should be kept approximately 10 F less than indicated for zero steel requirements because of the apparent sensitivity of crack widths to temperature in the cracking temperature range. Table 6.3.1 is based on this criteria.

TABLE 6.3.1—DESIGN TEMPERATURE LIMITS FOR UNREINFORCED CONCRETE WALLS (FOR LIMITING CRACKS TO 0.009 IN.)

Contraction joint spacing ft	Coefficient of thermal expansion × 10^{-5}			
	4	5	6	7
100	30 F	24 F	20 F	17 F
60	37 F	30 F	25 F	21 F
40	44 F	35 F	29 F	25 F
20	62 F	50 F	42 F	36 F

6.3.2 *Discontinuous external or end restraint*—Cracking will occur when the stress induced in the concrete by volume change exceeds the tensile strength of concrete as described in Section 2.3.3. When more than one crack is required in order to control crack widths, the total force in the reinforcing steel must equal the total restraint force induced at ends of the member as given in Eq. (2.6) of Section 2.3.1.

More than one crack will be required when the permissible crack width is less than the volume change ($L_R C_T T_E$).

When the volume change is less than the permissible crack width, no steel is required for volume change except as may be required as minimum (see Section 6.4).

6.3.3 *Internal restraint*—For relatively large masses the spacing of surface cracks will be controlled by internal restraint as described in Section 2.4. These cracks, independent of external restraint conditions, are not deep enough to require more than nominal amounts of reinforcing near the surface to control crack widths. In the example given in Section 2.4 the surface tensile stress due to daily temperature fluctuations was more than the surface stress due to the annual change in temperature. The depth of tensile stress block for the daily temperature fluctuations was less than 2.5 ft in the example. If this is assumed as the maximum depth of the critical restraint plane for internal restraint, then a maximum surface crack spacing in large masses of approximately 5 ft can be expected. If $C_T T_E$, using the maximum normal daily temperature fluctuation for T_E, is less than $w/12L'$, for L' in ft, then no surface reinforcement is required (Note L' should not be taken as more than 5 ft). If $C_T T_E > w/12L'$ then the minimum steel requirements of Section 6.4 should probably be utilized.

When internal restraint results from exposure of projecting elements from warm interiors, such as slabs projecting through exterior walls or walls projecting out of the ground, determine the depth of the tensile stress block and restraint factor as outlined in Section 2.4.1. If the required average crack spacing is less than twice the depth of the tensile stress block determine the size of bars to be distributed throughout the tensile stress block by:

$$A_b = \frac{1}{3}\frac{f_t'}{f_s}\frac{Bd_s}{N_H} \qquad (6.10)$$

where:

N_H = the total number of bars distributed throughout d_s

6.4—Recommendation for minimum reinforcement

The minimum requirements of ACI 318-71 should apply to all superstructure slabs and beams. The minimum total quantity of temperature and shrinkage reinforcement otherwise recommended for walls, slabs, and footings less than 48 in. thick which have been investigated for crack control by the measures outlined herein is 0.0015 times the cross sectional area A_g of the member. Not less than one half nor more than two thirds of the total quantity should be placed in any one face. For crack control the maximum bar spacing should be limited to 12 in. on center. For members more than 48 in. thick the minimum temperature and shrinkage requirements in each face should be limited by depth of cover (d_c) and bar spacing such that:

$$A_s' = \frac{f_t'A}{f_s} \text{ or } \frac{A}{100} \text{ (as a limit for } f_t'/f_s\text{)} \qquad (6.11)$$

The minimum bar size and spacing for members of this size should not be less than #6 bars at 12 in. on center.

No minimum temperature and shrinkage reinforcement is required for members 6 ft or more in thickness which are constructed by the principles and practices of Reference 1 to control the cracking of mass concrete provided the environmental conditions are such that cracking from internal restraint as discussed in Section 6.3.3 is not significant.

6.5—Design procedure

The basic procedure for problem solving is:

1. Determine the maximum effective temperature as outlined in Section 3.5
2. Determine the restraint characteristics of the element or structure as outlined in Chapter 2
3. Determine the physical properties of the concrete; tensile strength, elastic modulus, and coefficient of thermal expansion as outlined in Chapter 4
4. Determine the allowable maximum crack width from Section 5.2 or by some other established criteria
5. Determine the area of steel required to maintain cracking at the acceptable level
   (a) For members subject to continuous external edge restraint determine the required average crack spacing for the height of slab or height intervals of 5 to 10 ft above the base of walls as per Section 6.3.2. Where the required crack spacing is less than the spacing of joints provide reinforcement as per Section 6.3.2. In lieu of this the reinforcement in walls may be proportioned using Fig. 6.2 through 6.5. When the element or member is of sufficient size to require unreasonable amounts of reinforcement then additional measures will be required to control volume changes as recommended in Reference 1
   (b) For discontinuous external or end restraint, reinforcement will be required if: $w \leqq LC_T T_E$. If the member is subject to flexure determine the change in steel stress as per Section 6.2. If not determined the steel requirements as per Section 6.3.2
   (c) For members subject to internal restraint provide reinforcement as per Section 6.3.3 if the required average crack spacing is less than twice the depth of the tensile stress block

The following example problems illustrate this design procedure.

Example 6.1—Basement wall of power plant 30 ft high by 200 ft long is to be designed to retain backfill as a cantilevered wall for construction conditions. The wall is subject to ground water for its full height, with base slab on rock. It will be placed in 80 F ambient temperatures. Minimum final or operating air temperature will be 50 F. Design for limited leakage by limiting crack width to 0.009 in. and determine required wall thickness and reinforcement for the following conditions:

(a) Design for 3000 psi at 28 days and use the 5 bag mix of Example 3.2
(b) Same as (a) except contraction joints spaced 67 ± ft apart
(c) Design for 2000 psi at 28 days using mix of Example 3.1, no contraction joints and concrete cooled to 60 F placing temperature

6.1 (a)

Step 1.1—The minimum thickness of the wall at the base is 40 in. based on shear requirements for 3000 psi concrete. Assume wall tapers to a maximum thickness of 18 in. at the top

Step 1.2—V/S ratio (assume 10 ft lifts and wooden forms). Average thickness for first two lifts = 3 ± ft

$$V/S = \left(\frac{3\,(10)}{2\,(10) + 3}\right)\frac{3+4}{3} = \frac{70}{23} = 3.1 \text{ ft} \pm$$

Step 1.3—Assuming a placing temperature of 90 F without cooling measures, the effective placing temperature using Fig. 3.6 is 84 F ±

Step 1.4—The final temperature using formula 3.3 is 54 F

Step 1.5—The temperature rise following Example 3.2 is 52 F

Step 1.6—The design temperature equals 84 + 52 − 54 = 82 F

Step 2—Restraint (Fig. 2.1)

Step 3—Physical properties, f_c' @ 6 months = 5000 psi $\therefore f_T' = 6\sqrt{5000} = 420$ psi, assume $C_T = 5 \times 10^{-6}$ in./in./deg F and $f_T'/E_c = 105 \times 10^{-6}$ in./in.

Step 4—Limiting crack width = 0.009 in.

Step 5(b)—$f_s = 22$ ksi for 2½ in. cover and 12 in. spacing of bars from Eq. (6.2). Using Fig. 6.2 and Fig. 6.3, maximum temperature and shrinkage reinforcement is required for full height of wall for average thickness of 33 in.

h (ft)	B (in.)	K_R ($L = 200$ ft) Fig. 2.1	L' 6.4*	$L'/2h$	A_b 6.6*	A_b Fig. 6.3	Reinforcement
5	40	0.90	1.9	0.19	1.46	1.44	#9@8
10	36	0.84	2.1	0.10	1.45		
15	33	0.80	2.3	0.07	1.38		
20	29	0.75	2.5	0.06	1.23		
25	25	0.68	3.0	0.05	1.04		#9@12
30	22	0.62	3.5	0.05	0.92		

*Formula

Concrete = 537 cu yd @ $35 .... $18,800
Temperature reinforcement = 30 tons @ $400 12,000
Cost (excluding forms) $30,800

6.1 (b)

Everything same as (a) except $L = 67$ ft

From Fig. 6.2; maximum steel required only for the first 20 ft

h (ft)	B (in.)	K_R ($L = 67$ ft) Fig. 2.1	L' 6.4*	$L'/2h$	A_b 6.6*	A_b Fig. 6.3	Reinforcement
5	40	0.80	2.2 ft	0.24	1.36	1.35	#9@9
10	36	0.62	3.3 ft	0.18	1.31		
15	33	0.45	6 ft	0.23	1.14		
20	29	0.27	100 ft	1+	0.26		
25	25	0	—	1+	0.23	Minimum steel	#4@12
30	22	0	—	1+	0.20		

*Formula

Temperature reinforcement = 21 tons @ $400 .... $8,600

Note savings in reinforcing steel of $3400 to be weighted against the cost of two joints and added construction time.

6.1 (c)

Step 1.1—The minimum thickness of the wall at the base is 48 in. based on shear requirements for 2000 psi concrete. Assume wall tapers to 18 in. at the top.

Steps 1.2-1.6—The design temperature equals $69 + 15 - 54 = 30$ F.

Steps 2-4—Assume same as (a).

Step 5(b)—From Fig. 6.2 and 6.3 maximum steel ratio equals 0.003 for first 20 ft of wall.

h (ft)	B (in.)	K_R ($L = 200$ ft) Fig. 2.1	L' 6.6*	$L'/2h$	A_b (Min. steel)	A_b Fig. 6.3	Reinforcement
5	48	0.90	17 ft	1+	0.43	0.70	#6@9
10	43	0.84	24 ft	1+	0.39		
15	38	0.80	33 ft	1+	0.35		
20	33	0.75			0.30		
25	28	0.68			0.25	0.24	#4@10
30	25	0.62			0.23		

*Concrete = 612 cu yd @ $35† $21,400
Temperature reinforcement = 15.5 tons @ $400 6,300
$27,700
Savings in stress steel 3 tons for 8 in. additional depth 1,200
Net cost (excluding forms) $26,500

†Cost of cooling concrete assumed equal to savings in cement costs. Note example (c) is $4300 less than example (a) for the same design requirements.

Example 6.2—Culvert roof 36 in. thick supporting 20 ft of fill, spanning 20 ft between 4 ft thick walls 20 ft high by 100 ft long resting on a rock base, placed in 80 F ambient air, minimum final air temperature 20 F, no cooling of concrete, mix same as Example 3.1, stress steel #9@10 stressed to 24,000 psi in bottom face.

Step 1.1—The volume to surface ratio

$$(v/s) = \left(\frac{3\,(20)}{2\,(20 + 3)}\right)\frac{3 + 2}{3} = 2.2 \pm \text{ft}$$

Step 1.2—Effective placing temperature $= 90 - 0.6\,(10) = 84$ F (Using Fig. 3.6.)

Step 1.3—Final temperature is

$$20 + \frac{2\,(60 - 20)}{3}\sqrt{\frac{36}{96}} = 36 \text{ F}$$

Step 1.4—Temperature rise. The 15 F rise of Example 3.1 will be altered by the difference between a v/s of 2.2 and placing temperature of 84 F (30 F from Fig. 3.5) versus @ v/s of 2.7 in Example 3.1 and placing temperature of 69 F (25 F) or temperature rise $= 30/25\ (15) = 18$ F.

Step 1.5—Design temperature $= 84 + 18 - 36 = 66$ F.

Step 2—Restraint for end supports: (Eq. 2.2)

$$\frac{A_R h^3}{4 L_b I_c} = \frac{(1)\,(3)\,(20)^3}{4\,(20)\,(1)\,(4)^3/12} = 56 \pm$$

$$\therefore K_R = \frac{1}{1 + 56} = 0.0175$$

Step 3—$f_T' = 405$ psi $C_T = 5 \times 10^{-6}$ in./in./deg F

Step 4—Assume $w = 0.013$ in., $t_b = 2$ in. $\therefore At_b = 125$ for # 9 bars @ 10 in. o.c.

$$\therefore f_s = \frac{0.013 \times 10^3}{0.076\,\sqrt[3]{125}} = 34.3 \text{ ksi allowable}$$

Step 5—Steel requirements: (Eq. 2.4)

$$p = \frac{1.20}{12\,(33)} = 0.003,\ n = 9,\ j = 0.94,\ h = 20 \text{ ft}$$

$$K_f = (3)^3/20 = 1.35,\ K_c = (4)^3/20 = 3.2$$

$$\Delta f_s = \frac{0.0175\,(5 \times 10^{-6})\,(66)\,(29 \times 10^6)}{2\,(0.003)\,9\,(0.94)}$$

$$\left(\frac{20}{3}\left[\frac{1.35}{1.35 + 3.2}\right] + 4\ [0.003]\ 9\ [0.94]\right) = 6800 \text{ psi}$$

$$\therefore f_s = 24{,}000 + 6800 = 30{,}800 \text{ psi}$$

Note: This is less than allowable of 34,300 psi therefore no additional steel is required for volume change in the stress direction.

6.2 (a)

For the roof slab of Example 6.2 find the temperature steel parallel to the wall. Assume 3½ in. cover to center of temperature steel or $f_s = 26{,}000$ psi for bars at 12 in. spacing.

Note: Since the temperature rise of the slab is only 18 F the wall does not offer enough restraint to crack the slab therefore design the slab as an extension of the wall with a design temperature drop of 66 F.

Step 2—Restraint @ 5 ft from the wall for $L/h = 100/25 = 4$, $K_r = 0.40$

$$\text{Step 5—}L' = \frac{0.65\,(0.013)\,(10^6)}{12\,[0.4\,(5)\,(66) - 105]}$$

$$\therefore \frac{L'}{2h} = 0.52$$

$$A_b = \frac{0.4\,(405)}{26{,}000}(1 - 0.52)\frac{(36)\,(25)\,(12)}{50}$$
$$= 0.65 \text{ in./ft. \#6@8 in. o.c.}$$

Note: Less steel is required at the center, however, #6@8 is only slightly higher than minimum by ACI 318-71 therefore it should be used throughout the entire slab.

Example 6.3—A 6 ft thick power plant base slab supporting widely spaced walls. Construction joints but no contraction or expansion joints. Assumed placed in 75 F average ambient air temperatures with final unheated interior of 50 F. Slab is designed for operating uplift conditions requiring #11 bars at 12 in. o.c. stressed to 24,000 psi.

(a) Assume same concrete mix and conditions as Example 3.2

(b) Assume same concrete mix and conditions as Example 3.1

6.3 (a)

Step 1 — $v/s = 0.75$ (6 ft) = 4.5 ft maximum

(a) Effective placing temperature using Fig. 3.6
$T_{PE} = 85\text{ F} - 0.4\ (10) = 81\text{ F} \pm$

(b) Temperature rise using Fig. 3.5
For $v/s = 4.5$ @ $T_{PE} = 81$ F; temperature rise = 39 F
For $v/s = 2.7$ @ $T_{PE} = 81$ F; temperature rise = 31 F
∴ Net temperature rise = 39/31 (52 F) = 65 F

(c) Final temperature using Eq. (3.3)
$T_F = 50 + \frac{2}{3}\ (60 - 50)\ \sqrt{54/96} = 55\text{ F}$

(d) Design temperature
$T_E = 81\text{ F} + 65\text{ F} - 55\text{ F} = 91\text{ F}$

Step 2—Restraint (Fig. 2.1)
Without contraction of expansion joints the length is unspecified therefore assume L/H is greater than 20 or $K_R = 0.9$ maximum

Step 3—Physical properties, $f_T' = 6\sqrt{4600} = 405$ psi

Step 4—Limiting crack width = 0.013 in.
For bars @ 12 in. o.c. and cover of 2½ in. the allowable steel stress from Eq. (6.2) is 32,200 psi.

Step 5—Steel requirements

$$L' = \frac{(0.013)}{18\ [0.9\,(5)\ (91) - 105]\ 10^{-6}} = 2.4\text{ ft} \quad (6.4)$$

$$A_b = \frac{0.20\,(405)}{32{,}000}\left(1 - \frac{2.4}{12}\right)\frac{12\,(72)}{1} = 1.73\text{ in.} \quad (6.7)$$

Check

$\triangle f_s$ for flexure (2.5)

$\triangle f_s = 2\,(0.9)\ (5 \times 10^{-6})\ (91)\ (29 \times 10^{6}) = 23{,}800$ psi

$\Sigma f_s = 24{,}000 + 23{,}800 = 47{,}800$ psi

Since combined stress is greater than the allowable additional steel is needed, however, maximum steel requirements will be less than 1.56 + 1.73 = 3.29 in./in. or #11@6 in. o.c. Assume final bar spacing of 7 in. o.c. for an allowable steel stress of 38,500 psi. $A_s = 1.56\left(\frac{24}{38.5 - 23.8}\right) =$ 2.55 in./1; #11@7 ∴ OK.

6.3 (b)

For Example (b) the design temperature would be 32 F and $\triangle f_s = 8300$ psi so that combined stress equals 32,300 psi which equals the allowable, therefore no additional steel is needed for temperature.

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APPENDIX—METRIC CONVERSIONS

GENERAL CONVERSION FACTORS

From	to	Multiply by
in.	cm	2.54
ft	m	0.3048
in.^{2}	cm^2	6.451
in.^{3}	cm^3	16.387
ft^2	m^2	0.0929
yd^3	m^3	0.7646
lb	kg	0.4536
ton	kg	907.2
kip	kgf	453.6
lb/yd^3	kg/m^3	0.5933
lb/ft^3	kg/m^3	16.02
lb/in.^{2} (psi)	kgf/cm^2	0.07031
kip/in.^{2} (ksi)	kgf/cm^2	70.31
in./in./deg F	cm/cm/deg C	1.8
deg F	deg C	$t_C = (t_F - 32)\ /1.8$

Specific conversions

Customary	Metric
Section 3.2	
5×10^{-6} per F	9×10^{-6} per C
Eq. (3.2)	$T_{DS} = \left(16.7 - \frac{0.44V}{S}\right)\left(\frac{Wu - 74}{59}\right)$
Section 3.3	
Eq. (3.3)	$T_{min} = T_A + \frac{2(T_M - T_A)}{3}\sqrt{\frac{V/S}{244}}$
Section 4.3	
$\sqrt{f_c'}$ in psi	$0.265\sqrt{f_c'}$ in kgf/cm^2
Section 4.4	
$w^{1.5}33\sqrt{f_c'}$ in psi	$w^{1.5}$ (0.01368) $\sqrt{f_c'}$ in kgf/cm^2
$57{,}000\sqrt{f_c'}$ in psi	$15{,}100\sqrt{f_c'}$ in kgf/cm^2
Section 4.6	
6×10^{-6} per F	10.8×10^{-6} per C
7×10^{-6} per F	12.6×10^{-6} per C
Section 5.3	
Eq. (5.1)	$w = 1.085\sqrt[3]{d_cA}\,Rf_s \times 10^{-6}$ (f_s in kgf/cm^2)
Section 6.2	
Eq. (6.1)	$z = 1.79\,f_s\sqrt[3]{d_cA}$ (f_s in kgf/cm^2)
175 kips per in.	31,250 kgf/cm
145 kips per in.	25,850 kgf/cm
100 kips per in.	17,900 kgf/cm
Section 6.3	
Eq. (6.2)	$f_s = \frac{w \times 10^6}{1.085\sqrt[3]{d_cA}}$ in kgf/cm^2

This report was submitted to letter ballot of the committee which consists of 18 members; seventeen members returned ballots all of whom voted affirmatively and 1 ballot was not returned.

Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions

Reported by ACI Committee 207

ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction, and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be incorporated directly into the Project Documents.

Current methods available for evaluating physical properties of concrete in existing structures to determine its capability of performing satisfactorily under service conditions are identified and discussed. Although general knowledge of the structural design criteria used for the principal structures of a project is essential to determine satisfactory procedures and locations for evaluation of the concrete physical properties, analysis for the purpose of determining structural capability is not within the scope of this report. The report recommends project design, operation and maintenance records and in-service inspection data to be reviewed. Existing methods of making condition surveys and nondestructive tests are reviewed; destructive phenomena are identified; methods for evaluation of test and survey data are presented; and finally, preparation of the final report is discussed.

Keywords: alkali-aggregate reactions; alkali-carbonate reactions; cavitation; cements; chemical analysis; concrete cores; concrete dams; concrete durability; cracking (fracturing); dynamic tests; elastic properties; erosion; **evaluation**; extensometers; impact tests; inspection; laboratories; maintenance; **mass concrete**; nondestructive tests; nuclear power plants; post-tensioning; pozzolans; resurfacing; sampling: seepage; **serviceability**; spalling; static **tests**; stresses; surveys; x-ray diffraction.

Contents

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Chapter 1 — Introduction

The guidelines for evaluating the serviceability of concrete described herein apply to massive structures such as dams or hydraulic structures, bridge foundations and piers, building and reactor foundations.

Engineering Foundation Conferences on "Inspection, Maintenance and Rehabilitation of Old Dams" and "The Evaluation of Dam Safety" held at Asilomar Conference Grounds, Pacific Grove, California, 23-28 September 1973, and November 28-December 3, 1976 focused on problems involved in evaluating existing dams. In 1972 Congress passed Public Law 92-367 authorizing the National Program of Inspection of Dams.[1,2] Enactment of Public Law 92-367 pointed up the need for review and establishment of practices for the evaluation of nearly 50,000 existing dams and the materials of which they are constructed. The U.S. Army Corps of Engineers has completed an inventory of dams in the United States and prepared a report for Congress as provided in the law. The report includes guidelines for inspection and evaluation of dams and recommendations for implementation of a national dam inspection program.[3]

Other guidelines have been or are being developed which will provide good tools for those who will want to use this report as a guide for evaluating concrete in existing massive structures. They are the ACI Committee 201 report on making a condition survey,[6] the ASTM Committee C-9 Standard C823 for examination and sampling of concrete,[10] and the work of ACI Committee 437 which is developing systematic procedures for strength evaluation of existing structures.

1.1 — Scope

This state of the art report will focus on practices used to evaluate concrete in existing massive structures for the service conditions to which they will be subjected. Design considerations, evaluation of existing operating records and past inspection reports, condition surveys, maintenance reports, in-situ testing, instrumentation, identification of destructive phenomena, and final evaluation of the concrete are principal subjects which will be covered. Although this report will be limited to the evaluation of the concrete in the structure, design computations for determining areas of maximum stress in order to locate structural cracks and the selection of areas for sampling the concrete will be considered. The adequacy of the foundation, capacity of hydraulic structures and such factors as uplift, horizontal and vertical movement, seepage and erosion will be considered only as they affect the durability, cracking and strength of concrete.

1.2 — Objective

The objective of this report is twofold: (a) to present current methods available for evaluating the capability of mass concrete to meet design criteria under service conditions, and (b) to detect the retrogression in physical properties of concrete which could affect the capability of the concrete to meet design requirements in the future.

1.2.1 *Design considerations — Effect on materials properties required* — Although the objective of this report as mentioned in the scope is to evaluate the material properties, and not the structural adequacy of the concrete, it is important to review design requirements and criteria used for the structures of the project prior to undertaking materials investigations. This review permits realistic planning of investigations. For example, strength, elastic properties and the condition of the boundary concrete particularly at the abutments are important in arch dams. However, in gravity dams strength may not be as important, but cracking, leakage, foundation uplift pressures, etc., will be of prime importance. Durability of the concrete is important in both types of structures.

1.2.2 *Other considerations* — Careful review of any instrumentation data and a visual inspection of the concrete in all accessible parts of the structures by experienced engineers are important parts of the evaluation of the concrete. As many operating features should be used as feasible during the inspection so that the concrete can be observed under a variety of loadings. Past photographs which could reveal changes in the condition of the concrete should be reviewed when available.

1.3 — Report

The prepared report should identify and evaluate properties of the concrete as they relate to the important design criteria of the project structures, but should not preempt the structural engineer's responsibility for determining if the structures of the project are meeting design requirements. Photographic and graphic presentation of investigation data should be utilized to a maximum practical extent. The report, properly prepared, is an essential tool for those charged with the final responsibility of determining the structural adequacy and safety of the project.

Chapter 2 — Review of preconstruction data, construction, operation and maintenance records

Arrangements prior to an inspection should be made to obtain or have access to all available records and data pertaining to the structure. Pertinent engineering data to be reviewed include design criteria and memoranda, construction progress reports, instrumentation records, operation and maintenance records and to the extent available, preconstruction data. Information on adjacent projects, additions, or modification which may affect a change in the original design conditions should also be reviewed.

2.1 — Preconstruction evaluation

Engineering data relating to design criteria, design site conditions, purpose of project, and construction planning and procedure should be collected and arranged for ease of information retrieval. Documents which are readily available can be assembled first. Data which are missing but deemed necessary for evaluation can be identified. A suggested list of data to be reviewed is as follows:

2.1.1 *Project description document*
2.1.1.1 For a hydroelectric plant, the Federal Energy Regulatory Commission (FERC) licensed application
2.1.1.2 For a nuclear plant, the Preliminary Safety Analysis Report (PSAR)
2.1.1.3 All formal and final completion reports
2.1.2 *Contract documents*
2.1.2.1 Contract documents: Technical specifications and drawings
2.1.2.2 As-built drawings
2.1.2.3 Original issue drawings
2.1.3 *Regional data*
2.1.3.1 Land use map showing location of structure and its relation to surrounding localities
2.1.3.2 Topographic map of site and drainage area
2.1.3.3 Geologic plans and sections
2.1.3.4 Climatological data
2.1.3.5 Seismic data
2.1.3.6 Reservoir volume versus elevation curve
2.1.4 *Site subsurface data*
2.1.4.1 Logs of borings
2.1.4.2 Geological maps, profiles and cross sections
2.1.4.3 Soils investigation, availability of test results
2.1.4.4 Foundation treatment reports
2.1.4.5 Water table elevation
2.1.4.6 Geohydrologic data
2.1.5 *Site surface data*
2.1.5.1 Control elevations
2.1.5.1.1 For buildings: Finished grade, basement, floors, roof, etc
2.1.5.1.2 For dams and spillways: Crest, maximum and minimum reservoir surface, outlet works, etc
2.1.6 *Drainage*
2.1.6.1 Details of drains in structure
2.1.7 Environmental
2.1.7.1 Temperatures: Maximum, minimum and mean daily
2.1.7.2 Precipitation, maximum and mean annual
2.1.7.3 Average humidity, and range
2.1.7.4 Number of sunny days
2.1.7.5 Exposure: To sulfates from manufacturing plant wastes or natural sources; to deleterious organic acids from sewage or coal or cinder storage piles; to deleterious atmospheric gases from nearby industry

2.2 — Design criteria

2.2.1 *Design memorandum*
2.2.2 *Values of intermittent loadings, wind, temperature impact, loads*
2.2.3 *For hydraulic structures: Hydrostatic and hydrodynamic loads*
2.2.4 *Type of analysis: static, dynamic*

2.3 — Concrete laboratory records

2.3.1 *Materials used*
2.3.1.1 Cement
2.3.1.1.1 Certified mill test records
2.3.1.1.2 Additional physical and chemical properties tests
2.3.1.2 Pozzolan
2.3.1.2.1 Certified test records
2.3.1.2.2 Physical and chemical properties
2.3.1.3 Aggregates
2.3.1.3.1 Type and source(s)
2.3.1.3.2 Gradation
2.3.1.3.3 Summary of properties as specified in ASTM C33
2.3.1.3.4 Results of tests for potential reactivity
2.3.1.3.5 Report of petrographic examination
2.3.1.4 Water
2.3.1.4.1 Mixing water quality tests
2.3.2 *Concrete records*
2.3.2.1 Mix proportions
2.3.2.2 Water-cement ratio
2.3.2.3 Slump
2.3.2.4 Unit weight
2.3.2.5 Temperature records
2.3.2.6 Records of strength tests
2.3.2.7 Admixtures or air-entraining agents used, percent air entrained.

2.4 — Batch plant and field inspection records

2.4.1 *Storage and processing of aggregates*
2.4.1.1 Stockpiles
2.4.1.2 Rinsing and finish screeens for coarse aggregate
2.4.1.3 Bins or silos
2.4.2 *Cement, pozzolan and admixture storage and handling*
2.4.3 *Forms*
2.4.3.1 Type and bracing, tightness of joints
2.4.3.2 Time interval for stripping
2.4.4 *Preparation and condition of construction joints*
2.4.5 *Mixing operation*
2.4.5.1 Type of batch plant
2.4.5.2 Type of mixing equipment and mixing time
2.4.5.3 Condition of equipment
2.4.5.4 Monitoring and control practices
2.4.5.5 Any unscheduled interruptions due to plant breakdown or weather
2.4.5.6 Any scheduled seasonal interruption
2.4.6 *Method of transporting concrete: Pumps, chutes, conveyor belts, trucks, etc*
2.4.7 *Method of placing concrete in forms, including vibrator types and number*
2.4.8 *Concrete protection*
2.4.8.1 Curing methods: Water ponding or spray; curing compounds; shading; starting time and duration

2.4.8.2 Hot weather protection
2.4.8.3 Cold weather protection

2.5 — Operation and maintenance records
2.5.1 *Operation reports*
2.5.1.1 Instrumentation data
2.5.1.2 Unusual loading conditions
2.5.1.2.1 Earthquake
2.5.1.2.2 Floods
2.5.1.2.3 Extreme temperatures (temporary and prolonged)
2.5.1.2.4 Operational failure
2.5.1.3 Change in operating procedures
2.5.1.4 Shutdown of all or parts of the system
2.5.1.5 Increased loads or loadings
2.5.2 *Maintenance records*
2.5.2.1 Location and extent
2.5.2.2 Type of maintenance
2.5.2.3 Dates of repair
2.5.2.4 Repair materials
2.5.2.5 Performance of repaired work

Chapter 3 — Review of in-service inspections

3.1 — General

The previous chapter outlined portions of the information which may be available on the history of a structure; primarily, that concerning preconstruction investigations, design criteria, actual construction records, and operations and maintenance records. In addition a considerable amount of history may also be obtained from service records, namely, that from in-service inspections.

Most organizations monitor the performance of completed structures to assure that they function safely and in accordance with the design whether they are massive bridge piers, nuclear containment vessels, or hydroelectric or flood control projects. The monitoring may be part of the owner's operation and maintenance program or may be required by law.[4,5] Service records are generally more complete for recently constructed structures than for older structures inasmuch as the concern for public safety has increased in recent years. The scope of surveillance can vary widely between organizations and may depend to an even greater extent on the size and nature of the project or structure and potential hazards it may present.

In order to properly compare and evaluate the existing condition of concrete in massive structures, it is essential to review these in-service records which may also include routine and periodic inspections.

3.2 — Routine inspections

Routine inspection by various organizations are generally made at a frequency of 6 months to 2 years. They commonly consist of a visual examination of the condition of the exposed and accessible concrete in various components of a structure or project. Submerged structures or portions thereof may be visually examined by diver. In some cases, visual examination may be supplemented by nondestructive tests as described in Chapter 5 to indicate certain properties and conditions of the in-situ concrete at that particular time, such as compressive strength, modulus of elasticity, and presences of voids and cracking. Data from instrumentation embedded in the concrete may also be available. A comparison of the concrete properties, conditions and instrumentation at each inspection interval are useful analysis tools and may point out abnormal changes.

Immediately after placing the structure in service frequent inspections should be made so that performance can be assessed and, if necessary, modifications can be made to the design and operating practices. Inspections made thereafter are directed at identifying any changes in condition of the concrete or concrete properties which may affect the integrity of the structure. Inspections may be performed by trained technicians or qualified engineers depending on the program established. A report describing the findings of each routine inspection should be prepared which notes any changed conditions, contains photographs of the conditions and recommends corrective action. Further in depth investigations should be initiated if for any reason problems are suspected. Documentation of the inspection and, if required, action taken normally should be filed with the owner.

3.3 — Periodic inspections

Periodic inspections are generally conducted at a frequency of 2 to 10 years and are the same in nature or objective as routine inspections. However, periodic inspections involve a more detailed study. Periodic inspections are generally associated with higher risk structures or projects and supplement the routine inspections. However, it should be emphasized that, unless changes in the outward appearance of the concrete or concrete structures are noted, extensive periodic inspections may not be necessary.

Periodic inspections may include considerable preparation such as dewatering or arranging means for inspecting submerged portions of a structure, excavating inspection trenches, and comprehensive review of instrumentation data, design and operating criteria, etc. as may be required for a complete evaluation. In addition the periodic inspection may include sampling of seepage waters, nondestructive testing, and determination of stress conditions. The amount of investigative work necessary usually depends on the condition of the concrete. It should yield sufficient detailed information to provide practical guidance for the selection of the best method of repair or maintenance work necessary. In some cases, the actual maintenance work may be accomplished at the same time as the periodic inspection.

The scope of the inspection should also include identification of causes of deterioration. Methods and techniques for performing investigative work in connection with periodic inspections will be discussed in detail in Chapters 4, 5, and 6.

Documentation of the inspections should be on file with the responsible authority.

3.4 — Inspection reports and records

The in-service inspection reports and records previously described are in essence a history of the project or structure from which future performance can be predicted. In addition to a qualitative description, the information presented may supply actual values which can be utilized in structural analysis and comparison with the original design.

Chapter 4 — Condition surveys

A condition survey is a visual examination of exposed concrete for the purpose of identifying and defining areas of distress and may include examination of interior concrete. Conditions are described in common terminology which can later be perceived by the Engineer. The appendix to the ACI Committee 201 report, "Guide for Making a Condition Survey of Concrete in Service," presents terms associated with the durability of concrete and a series of photographs typical of these conditions.[6] This should be reviewed prior to making a condition survey. ASTM C823, "Examination and Sampling of Hardened Concrete in Constructions," contains additional information useful in conducting a condition survey.

4.1 — Cracking surveys

4.4.1 *Scope* — A cracking survey is an examination of a concrete structure for the purpose of locating, marking, and identifying cracks, and of the relationship of the cracks with the other destructive phenomena. In most cases, cracking is the first symptom of concrete distress. Hence, a cracking survey is significant in evaluating the future serviceability of the structure. Some cracks may occur at an early age and may not be progressive; others may occur at later ages and increase in extent with time; and some may occur following some unusual event.[7,8]

4.1.2 *Procedure* — The initial step in making a crack survey is to locate and mark the cracking and define it by type. The ACI Committee 201 report, cited previously, should be consulted and utilized. Cracks are classified therein by direction, width and depth, using the following adjectives: longitudinal, transverse, vertical, diagonal, and random. The three width ranges suggested are: fine — generally less than 1 mm; medium — between 1 and 2 mm; and wide — over 2 mm. Width and depth can normally be determined using an average of feeler gage readings or by readings from a suitable measure or pocket comparator. Highly accurate determination of crack width can be obtained with a commercially available hand-held illuminated microscope with internal scale divisions of 0.02 mm. When a series of measurements are to be made over a period of weeks or months, the measurement point location should be marked and the sharp edges of the crack protected by a thin coat of clear epoxy to avoid breakage. If possible, the depth should be determined by observing edges or inserting a fine wire or feeler gage; however, in most situations the actual depth may be indeterminable without drilling or use of other detection techniques such as the pulse velocity described in Chapter 5.

The nature of the cracking should be defined in common terminology which can be visualized by others less familiar with the structure. These terms, in addition to the classifications previously mentioned, should also include such visual cracking terminology as, pattern cracking, surface checking, hairline cracking, and D-cracking.

Conditions which may be associated with the cracking either over portions of the length or for the entire length should be noted. These conditions may include seepage through the cracks, deposits from leaching or other sources, carbonation of surfaces adjacent to cracks, spalling of edges, differential movement (offsets), etc. Chemical analyses of the seepage water and the deposits may be desirable.

It may be worthwhile to repeat the survey under various loading conditions when change in crack width is suspected. Furthermore, tapping of surfaces with a hammer may detect shallow cracking beneath and parallel to the surface. A hollow sound generally indicates that such cracking is likely even though it cannot be seen.

4.2 — Surface mapping

4.2.1 *Scope* — Surface mapping may consist of detailed drawings produced from hand mapping, photographic or movie film mapping, or a combination of these or similar techniques. Surface maps become permanent record of the condition of the concrete at the time each survey is made and are an integral part of the report. Items most often identified and mapped include: cracking, spalling, scaling, popouts, honeycombing, exudation, distortion, unusual discoloration, erosion, cavitation, seepage, conditions of joints and joint materials, corrosion of reinforcement (if exposed), and soundness of surface concrete.

4.2.2 *Procedure* — A list of items recommended for surface mapping by hand is as follows:

(a) Structure drawings, if available.
(b) Clipboard and paper or field book.
(c) Tape measure, 50 to 100 ft (15 to 30 m).
(d) Ruler graduated in 1/16 in. or mm.
(e) Feeler gage.
(f) Pocket comparator or hand microscope.
(g) Knife.
(h) Hammer — 2 lb (1 kg).
(i) Fine wire (not too flexible).

(j) String.
(k) Flashlight or lantern.
(l) Camera w/flash and assortment of lenses.
(m) Assortment of film — color and high speed.

Mapping should begin at one end of the structure and proceed in a systematic manner until all surfaces are mapped. Both external and internal, surfaces should be mapped if access is available. Use of 3-dimensional isometric drawings is occasionally desirable showing offsets or distortion of structural features. Areas of significant distress should be photographed for later reference. A familiar object or scale should be placed in the area to show the relative size of the area included. It is important to describe each condition mapped in clear, concise detail and avoid generalizations unless it is common to other areas previously detailed. Profiles are advantageous for showing the depth of erosion.

4.3 — Core drilling and testing

Core drilling is presently the accepted method of obtaining information on concrete within the structure in areas which otherwise can not be observed. However, core drilling is expensive and should only be considered when sampling and testing of interior concrete appears to be necessary.

The presence of abnormal conditions of the concrete at exposed surfaces may suggest questionable quality or a change in the physical or chemical properties of the concrete. These conditions may include scaling, leaching, pattern cracking, and freeze-thaw weathering, to name the most common. When such observations are made, core drilling to examine and sample the hardened concrete may be necessary. The minimum depth of sampling concrete in massive structures should be 2 ft (0.6 m) in accordance with ASTM C823. However, under some conditions, core drilling of the entire thickness may be required to obtain representative samples of a monolith. Occasionally, this drilling can be coordinated with foundation exploration work.

The diameter of core holes will depend on the testing anticipated. For compressive strength, static or dynamic modulus of elasticity, or similar laboratory tests, the diameter of the core should be between 2.5 to 3.0 times the maximum size of aggregate. However, 8 or 10-in. (200 or 250 mm) diameter cores are generally drilled for 6-in. (152 mm) MSA concrete because of the higher cost and handling problems of larger diameter cores.

Cores obtained from drill holes should be logged by methods similar to those used for geological subsurface exploration. Logs should show, in addition to general information on the hole, conditions at the surface, depth of obvious deterioration, fractures and conditions on fractured surfaces, unusual deposits, coloring or staining, distribution and size of voids, locations of observed construction joints, and contact with the foundation or other surfaces. See Section 6.1 for additional instructions on the examination of cores.

Cores recovered from drilling operations should be immediately marked for identification, including location, depth, and notation of the top and bottom, and should be placed in protective core boxes or protective wrapping. They should then be stored in safe areas protected from the weather.

4.3.1 *Strength and elastic property determination*

4.3.1.1 *Standard tests.* The following ASTM test procedures should be used for determining physical properties of drilled concrete cores:

C42 — Obtaining and Testing Drilled Cores and Sawed Beams of Concrete for compressive strength and tensile strength

C215 — Fundamental Transverse, Longitudinal and Torsional Frequencies of Concrete Specimens for dynamic modulus of elasticity (Young's Modulus)

C469 — Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression for static modulus of elasticity and Poisson's ratio

4.3.1.2 *Other tests.*

4.3.1.2.1 *Dynamic loading.* This generally refers to a load application time, or a complete tension-compression loading and unloading cycle, which is a fraction of that normally used or experienced in conventional static beam or cylinder testing. Examples might be the forces generated by blasts, explosions, or earthquakes. Tests have indicated concrete shear strengths are 50 percent and 80 percent greater under single pulse dynamic loading rates equivalent to 7 Hz and 300 Hz than under static load rates.[22,23] Little data exists on the magnitude of possible tensile strength increases for comparable loading times.

4.3.1.2.2 *Seismic loading.* Earthquake or seismic loading is at the lower end of the dynamic range, or a total tension-compression cycle period between 1 Hz and 10 Hz. This is equivalent to single-mode load application rates from 0.25 to 0.025 sec. Direct tensile tests at these rates indicated no increase in concrete strengths above static rate levels for dry concrete, but a 30 percent increase for moist concrete.[24] Other tests show an increase in compressive strength of from 30 percent to 50 percent with an increasing loading rate within the seismic range.[25] Of possible significance is the lower rate of strength increase by older (1 year or more) concrete, where the maximum increase under rapid loading was less than 25 percent.

4.4 — Prototype seismic load tests

This type of loading is accomplished in the field using either forced (artificial) or ambient vibration. In the forced vibration technique the mass is vibrated at known frequencies and mode shapes, frequencies and damping effects are measured at various locations in a structure. Similar observations are also made using natural vibrations induced by wind, wave action, and micro seismic loading.

4.5 In-situ stress determinations

In evaluating the effects of observed distress due to materials deterioration, excessive dynamic or static loading, and other causes, determination of existing stress conditions may be necessary to assure safety of the structure. In-situ stress determinations have been primarily limited to arch dams where stress analysis may be complex, and, in some instances, structural movements in service change the pattern and distribution of stress considered in the original design. Stress conditions determined can be compared with design parameters and with existing strength levels. One method which has been successfully used to investigate in-situ stress conditions is the "Over Coring-Stress Relief" Method.

4.5.1 *Over coring* — The over coring technique was originally developed in the study of rock mechanics. However, in the last 10 years it has also been applied to investigate the in-situ stress in concrete structures. The U.S. Bureau of Reclamation used the over coring stress relief method to investigate three thin arch dams located near Phoenix, Arizona.[15,31] The procedure involved drilling an EX size hole, 1-13/16 in. (45 mm) nominal diameter; gage; over coring the EX hole with a 6 in. (152 mm) core barrel; and recording the strain at 60 deg intervals around the circumference of the gage. Drilling three horizontal holes, which intersected near the center of the structure and at an angle of 22.5 deg with each other, produced accurate determinations of in situ maximum and minimum stress conditions. The results further showed that in arch dams, a single drill hole drilled approximately normal to the principal stresses in the vertical-tangential plane was adequate for maximum/minimum stress determinations. Accuracy of the results also depends, to a large extent, on good drilling equipment and techniques, and experienced crews. The borehole gage used was developed by the U.S. Bureau of Mines and was later modified for water-tightness and ease of maintenance. Modulus of elasticity at each measurement point was determined in the field using the 6 in. (152 mm) donut-shaped core taken from each location. A special apparatus was used to hydraulically load the core section in a chamber with a borehole gage inserted in the EX hole. The thickwall cylinder formula was used to compute the modulus of elasticity. The 6 in. (152 mm) overcore recovered was also tested for triaxial shear, compressive strength, tensile strength, modulus of elasticity, Poisson's ratio, specific gravity, absorption, alkali-aggregate reaction, and used for petrographic examinations.

4.5.2 *Other methods* — Two other methods of determining the in-situ stress conditions have been widely used in rock mechanics and could be applied to concrete.[16] These methods include the flatjack and the velocity propagation. The flatjack method involves cutting a slot in the concrete and provides a measure of actual stress in the surface plane. However, this method is restricted to near-surface measurements because of the difficulty of cutting deep flatjack slots. The velocity propagation method utilizes measurement of stress waves passed between two points. Accordingly, two or more bore holes enable crosshole wave measurements.

4.6 — Supplemental instrumentation

Supplemental instrumentation may be required when unusual behavior or changing conditions are detected during inspection of the structure. Conditions may relate to movement of the structure, movement within monoliths of the structure along joints or movement within monoliths at cracks. Other instrumentation may include equipment for measuring hydrostatic pressures in cracks and joints and under the structure (uplift). Instrumentation which has been found most valuable in evaluating existing structures is described in the subsequent subsections.

4.6.1 *Extensometer points* — An arrangement of three embedded plugs, two on one side of a crack or joint and the third on the other, will provide a measurement of relative shear movement as well as crack width change. A Whitmore strain gage or equivalent is used to measure the change in length between plugs.

4.6.2 *Borehole extensometers* — Primarily intended for measuring consolidation of weaker layers within rock, but can be used to detect internal movement at structural cracks.

4.6.3 *Electronic distance measuring instruments* — This commercially available equipment is capable of accuracies from 5 to 10 mm over distances up to 9 km, with adequate reflector targets, atmospheric corrections, and proper techniques. They are most useful for monitoring structure displacements.

4.6.4 *Joint meter* —The joint meters are embedded across joints or cracks to measure the opening and closing. Measurements can be taken at some remote location by connecting cable. Joint meters are commercially available from firms specializing in instruments for embedment in soil and concrete.[32]

4.6.5 *Electrolevel.* — This is a highly-refined spirit level, with the position of the bubble determined by means of electrodes. Changes in slope of 0.0005 in. per in. (50 millionths) can be measured, remotely if desired. The devices have been vulnerable to moisture corrosion.

4.6.6 *Cased inclinometer* — These are accelerometers housed in a wheeled probe which is passed through a grooved casing. Inclination from vertical is determined at selected elevations, with a sensitivity of one part in 10,000. This is a more precise version of the slope indicator equipment originally developed for monitoring subsurface movements in soils.

4.6.7 *Tilt-measuring instruments* — A portable accelerometer sensor mounted on a metal plate, placed upon reference plugs or plate embedded in the structure senses changes in rotation of the order of 10 sec of arc. This is comparable to the electrolevel precision.

4.6.8 *Observation wells* — Are simply open holes into the foundation in which water level measurements can be taken to determine uplift pressure at that location.

4.6.9 *Piezometer* — An instrument for measuring pressure head. Generally, the piezometer consists of a pressure cell installed in a drill hole in the foundation.

4.6.10 *Vertical and horizontal control* — Survey points for line and level measurements are established at various locations on the structure for the purpose of measuring differential movements with time. History plots of data, covering months or years, may be necessary to differentiate between normal and extreme or critical movements. Data may reveal cycles associated with temperature or applied loading. Whenever possible, estimated values of deformation or displacement should be developed, based on theoretical analyses using the best available data on materials, properties and parameters. Observed values may indicate distress when the expected or normal movements are exceeded.

4.7 — Geophysical logging

Several geophysical drill hole logging techniques often used in the oil industry are available and may be utilized to provide supplemental data on the physical properties and condition of in-situ concrete.[9] Geophysical logging consists of lowering various instruments into an open drill hole; the type of instrument dependent on the type of measurement (log) to be developed. As the instrument is lowered to or withdrawn from the bottom of the hole, an automatic recorder traces the log on graph paper. The recorder paper on which the log is traced moves on a vertical scale with the instrument and measurements received from the instrument are plotted on the horizontal scale. In general, porosity and density are the most common parameters derived from geophysical logs. Porosity may be determined from several logs including Sonic, Density, and Neutron Logs. Density can be directly obtained from the Density Log. Also, the previously mentioned logs together with Resistivity and Caliper Logs provide a graphic record of the uniformity of concrete throughout the depths examined. When drill hole core recovery is poor or is not practical, geophysical logging can provide a method of locating cracks, voids, contacts and other discontinuities of significance. Logging of drill holes and interpretation of logs should be done by firms which specialize in this exploration technique.

4.8 — Down hole TV camera

The condition of interior concrete and foundation rock can be examined directly, and video-taped if desired, by use of small transistorized TV cameras. These instruments are successors to the Corps of Engineers borehole camera which is no longer generally available. The TV cameras are housed in 6 in. (150 mm) and 3 in. (75 mm) diameter probes, capable of remote focusing and aiming axially as well as in a radial direction. The transmitted picture is continuously displayed on a scanner screen, and can be supplemented by video recording for a permanent record. The camera assembly will resist hydrostatic heads up to 1300 ft (400 m) and the focusing capability will permit estimating the size of caverns or cavities encountered. Turbidity of the water must be controlled for best results. Both the Bureau of Reclamation and Corps of Engineers have used this technique with satisfactory results.[29]

4.9 — Seepage

Seepage is the movement of water or other fluids through pores or interstices. Some structures may include design features to safely control seepage such as waterstops, sealed joints, cut off walls, grout curtains, granular drains and drainage galleries. These features should be checked to assure they are functioning as designed. Seepage can be important with respect to durability, can indicate failure of the structure to function monolithically and may also indicate operating problems in water retention structures. Seepage occasionally occurs through unbonded horizontal or vertical construction joints; around waterstops or sealants in expansion, contraction or control joints; along cracks; along the interface between concrete and some other material such as foundation contacts, form bolt or tie holes, or other embedded items; or through areas of porous low quality concrete.

Water from seepage may result in the development of excessive hydrostatic heads on portions of the structure, may attack the concrete chemically, provide excess moisture to produce mechanical failure during freeze-thaw cycles, or may transport undesirable particles from the concrete or foundations. Analysis of seepage water can be used to evaluate chemical activity. The appearance of seepage water, whether clear or cloudy, will indicate the presence of transported sediments or dissolved minerals. Determination should also be made of the extent and the quantity of seepage water if measurable.

Frequently, it is important to know the source and velocity of seepage. The source can sometimes be obtained by simple measurements comparing the temperature of seepage with groundwater or reservoir temperatures. Dye tests can be made utilizing commercial dyes such as Rhodamine B (red) or Fluoroscein (green) both of which are acceptable by the DEQ (Department of Environmental Quality). The dye is introduced into water at some location near the upstream face, in drill holes, or other appropriate accessible points. The location and time of reappearance will indicate the source of various seeps and will provide the velocity of dye movement.

Features designed to control seepage should be checked to assure they are in good conditions and functioning properly.

4.10 — Surface damage

Surface damage may be caused by cavitation, impact, abrasion, freeze-thaw deterioration, chemical attack, etc. A survey of such damage should provide information on the area affected, depth, and its nature. Sections and profiles utilizing surveying techniques are valuable in evaluating the extent and depth of erosion. Notation of evidence in the areas of damage commonly provide keys to diagnosing the cause. Such evidence may be loose, semi-detached fragments, D-cracking, rock and debris piles, offsets or protrusions, coloration, and overall condition of the damaged area and of the surrounding concrete. These observations should be recorded.

During routine inspections only exposed surfaces are generally surveyed. However, for periodic inspections or for special observations deemed necessary during routine inspections, surfaces flooded, under water, or backfilled and underground should be checked for surface damage by various methods. The method selected may depend on the size and depth of the area to be surveyed, conditions in the area, including water depth, and whether maintenance work will be done at the time of the inspection. Usual methods used include excavation, dewatering the structure, observation by submerged closed circuit television camera mini-submarine inspection, diver inspection, and sounding. Dewatering or excavation are usually the most expensive and, therefore, are generally done only when there is concern about safety of the structure.

Failure to properly identify and correct surface damage can result in excessive wear or cavitation resulting in loss of the design hydraulic characteristics, mechanical equipment malfunction and, in extreme cases, the loss of structural stability.

4.11 — Joint survey

Joints in massive structures should be surveyed to assure they are in good condition and functioning as designed.[12] Location and type of each joint, whether expansion, contraction, or construction, should be noted together with a description of its existing condition. Joint openings should be measured under various loading conditions if appropriate. The joints should be carefully examined for spalling or D-cracking, absences or presence and condition of joint fillers, and evidence of seepage, emission of solids or chemical attack. Measurements should also be taken of surface offsets on either side of the joints or other irregularities. Joint construction details should be recorded and mapped if drawings are not available.

Chapter 5 — Nondestructive testing

5.1 — Scope

The purpose of nondestructive testing is to determine the various properties of the concrete such as strength, modulus of elasticity, homogeneity, integrity, as well as conditions of strain and stress without damaging the structure. Selection of the most applicable method or methods of testing will require good judgment based on the information needed, size and nature of the project, and the seriousness of observed conditions.[13,26] In-situ testing, if required, normally should follow a condition survey. Generally determination of the concrete properties is only necessary to further evaluate the effects of observed distress on the safety or serviceability of the structure. In-situ testing will provide parameters for structural analysis by current analytical techniques for comparison with present day design requirements.

5.2 — Surveying techniques

5.2.1 *General* — Although compressive strength and modulus of elasticity, depending on the method used, can be estimated from the survey techniques described in the following subsections, the accuracy of these estimations are usually considered to be only relative based on the many factors which can influence the various measurements. The measurements of strength estimations may be greatly improved if they are correlated with test results on drilled core specimens from the same structure. The techniques described are valuable survey tools in that results provide comparative values. When surveys are made at different times, changed conditions can be detected and monitored.

5.2.2 *Schmidt hammer* — The Schmidt hammer, also referred to as a Swiss, rebound, or impact hammer, is a lightweight portable instrument used for qualitative measurement of in-place concrete strength. The greatest value of the hammer is for comparison of indicated strength between different areas, thereby detecting areas of potentially low strength. The indicated strength is recorded on a built-in scale which measures the rebound of a spring-driven plunger after it strikes the concrete surface. Rebound is affected by many factors such as the mix composition, aggregate properties, surface texture and curvature, moisture content, and mass of the concrete tested. Calibration by statistical correlation with the strength of cores drilled from the structure will indicate the degree of reliance that can be placed on strength estimated from rebound readings. Calibration on concrete test cylinders is helpful in estimating strength or relative differences in strength, but such estimates must be used with care. Use of published calibration data to estimate strength from rebound readings is of only limited value. However, the Schmidt hammer is an excellent tool for quickly determining the uniformity of concrete in-place. The method of testing concrete by the

Schmidt hammer is described in ASTM C805. No correlation has been found between rebound readings and modulus of elasticity.

5.2.3 *Windsor probe* — The Windsor probe method of test consists of driving a precision probe into concrete utilizing a "gun" which produces a specific energy. Generally, three probes are driven into the concrete at each location in a triangular pattern, controlled by template. The protruding ends of the probes are measured. The Windsor probe system has been found comparable with the Schmidt hammer. On concrete 40 to 50 years old, the probe system may yield higher strength than actually exists. Limited information suggests that the cause of higher indicated values may relate to microcracking between the aggregate and paste which are indicated by test cylinder results but not by the probe readings. As with the rebound hammer, interpretation of test results based on other known factors as discussed above, are necessary to effectively use this equipment. The Windsor probe test procedure is described in ASTM C803.

5.2.4 *Pulse velocity* — Pulse velocity testing involves measurement of the velocity of groups of compression waves through concrete. The method provides a quick means of assessment of the uniformity of in-place concrete and detection of deterioration or cracking, or both.[14] The test method is described in ASTM C597, "Pulse Velocity Through Concrete."

The equipment used is very portable consisting only of a lightweight instrument housing a pulse generator and receiver and high speed electronic clock, transmitting and receiving transducers, and cable connectors. Velocity is determined by dividing the measured wave travel time by the shortest direct distance or path length between transducers. When a signal cannot be received it usually indicates one of the following conditions: an open crack, insufficient consolidation, or the energy was absorbed between the transducers. Accordingly, pulse velocity equipment may be used in determining crack depth. Available equipment is effective up to a path length of approximately 50 ft. It is important that a high degree of accuracy is used in determining both travel time and path length since small errors in either measurement may produce significant changes in the indicated pulse velocity.

Velocity measurements are usually made between exposed surfaces with one transducer stationary while the other transducer is moved from point to point within an effective area. Measurements can also be made from inspection or drainage galleries within the structure if available and accessible. Pulse velocity surveys have had relatively wide usage as one of the techniques for quality investigation of existing concrete dams and other concrete structures.

5.2.5 *Other techniques* — A variation in the pulse velocity technique is a proposed pulse echo system wherein a compressional wave pulse is transmitted from, and its echo received back at, the same surface or same point. Reflection times from interfaces, cracks, or voids, together with the known velocity within the concrete, permit calculation of distances from the discontinuity to the transmitting and receiving point. The scheme has been demonstrated to be feasible, but is still under development.

Similarly still in a development stage, is an acoustical imaging system which is intended to produce a digital contour map or profile of the top and bottom surfaces of a submerged concrete surface, such as a dam stilling basin. The principle is borrowed from the acoustical holography procedure used to examine steel vessels and welds.

Chapter 6 — Identification of destructive phenomena

6.1 — Petrographic analysis

6.1.1 The petrographic analysis of concrete should be made by a person qualified by education and experience to operate the equipment used in the analysis and to record and interpret the results obtained. The petrographer should be consulted before samples are taken in the field and should be furnished with preconstruction, construction and condition reports described in Chapters 2, 3, and 4.

6.1.1.1 — Taking of samples of concrete for laboratory testing and analysis presents great problems of judgment in order that the samples are truly representative of the conditions to be studied. The surveys made under Chapters 2, 3, and 4 should furnish information for location and number of samples required. The most useful samples for petrographic examination of concrete are diamond-drilled cores with a diameter of at least twice, and preferably three times, the maximum size of the coarse aggregate in the concrete. If 6 in. (150 mm) aggregate was used, a core 8 to 10 in. (200 to 250 mm) in diameter has been found to be satisfactory and is commonly taken in practice to avoid the high cost and handling difficulty of 12 to 18 in. (305 to 457 mm) cores.

6.1.1.2 — Sampling should be done with complete objectivity, so that the suite of samples is not weighted with either the unusually poor or unusually unsound materials. In securing samples care should be taken to avoid disturbance or contamination of the materials to assure laboratory tests and analyses are truly representative. Coring is preferable to sampling by other means because the concrete is disturbed to a minimum. Use of sledges or air hammers may induce internal fracturing or may so disrupt the concrete as to make it difficult or impossible to describe its structure accurately and in detail.

6.1.1.3 — The sampling should include both near-surface concrete and concrete at depth, inasmuch as they may differ substantially in development of cracking, deterioration of the cement paste, progress of cement-aggregate reactions and other features. The samples should be sufficient in size and number to permit all necessary laboratory tests. The petrographic examination should be performed on concrete that has not already been subjected to a compression test or some other test.

6.1.1.4 — Visual inspection with the unaided eye, a hand lens and a stereoscopic microscope can provide valuable information when applied to original exterior surfaces, surfaces of fractures and voids, surfaces of fresh fractures, and through the cement paste and aggregate. From this examination the following features can be studied and described:

- Condition of the aggregate
- Pronounced cement-aggregate reactions
- Deterioration of aggregate particles in place
- Denseness of cement paste
- Homogeneity of the concrete
- Occurrence of settlement and bleeding of fresh concrete
- Depth and extent of carbonation
- Occurrence and distribution of fractures
- Characteristics and distribution of voids
- Presence of contaminating substances

As part of the visual examination, noteworthy portions of the concrete, secondary deposits, or particles of aggregate are separated for more detailed microscopical study or for chemical, x-ray diffraction or other types of analyses.

6.1.1.5 — Petrographic thin sections permit thorough examination of concrete because details of texture and structure are preserved. Such sections are slices of concrete that are cemented to a small glass plate and then are ground thin enough to readily transmit light. When so prepared, the sections can be examined under the petrographic microscope at magnifications up to about 1000 diameters, or with oil immersion objectives to about 2000 diameters. From the examination of thin sections the following features can be studied and described:

- Composition of fine and coarse aggregates
- Evidence of cement-aggregate reaction
- Proportion of unhydrated granules of cement
- Presence of mineral admixtures

6.1.1.6 — In some instances, petrographic methods other than microscopy, such as x-ray diffraction and differential thermal analysis, may be required or might most rapidly serve to identify fine-grained materials.

6.1.1.7 — Sawed and finely ground surfaces of concrete are used in microscopical analysis of concrete to determine the air content and various parameters of the air void system in accordance with ASTM C457. This method can also be used to analyze the concrete for the volumetric proportions of aggregate, cement paste, and air voids.

6.2 — Chemical analysis

6.2.1 — Although hardened concrete may be subjected to chemical analysis for any of many reasons, the most common is for determination of the proportion of cement used in the mixture. ASTM Method C85 and variants of this are usually employed for this purpose.

6.2.2 — Dependable quantitative chemical methods for detection of organic admixtures in hardened concrete have not been developed. Calcium chloride is the only commonly used admixture that can be quantitively determined by chemical methods. Substances formed by degradation of lignosulfonate in portland cement mixtures can be detected by characteristic fluorescence of water solutions produced by acid extractions of hardened concrete at ages up to 2 years. The method, although not quantitative, is sufficiently sensitive to indicate the presence of lignosulfonate in amounts equivalent to less than 0.1 percent by weight of the cement. No generally-applicable methods are available for detection of the many other organic admixtures used in concrete.

6.2.3 — Concrete may contain any of a wide variety of organic or inorganic substances, either as contaminants in the concrete making materials or the fresh concrete, or because they were absorbed into the hardened concrete. Inorganic chemicals can be determined by classical analytical methods, but the results may be difficult to interpret when they are similar to chemicals that were deliberately included in the concrete. Organic substances are particularly difficult to identify. Evidence available at the jobsite might present the solution to problems of attack of aggressive chemicals upon the hardened concrete.

6.3 — Physical tests

Frost and freeze-thaw resistance of concrete specimens can be determined by ASTM Test Methods C671 and C666, respectively. Furthermore, results of the freeze-thaw tests may be useful in predicting the relative rate at which deterioration of concrete in the structure may occur and service life of the structure.

6.4 — Report

6.4.1 *Location and orientation of cores tested*

6.4.2 *List of physical and chemical tests and their results*

6.4.3 *Photographs of cores as received, photographs and photomicrographs of features of interest, and photomicrographs of thin sections*

6.4.4 *Conclusions based on test results of condition of concrete*

6.5 — Applicable documents

6.5.1 *ASTM standards*

C42, Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

C85, Cement Content of Hardened Portland Cement Concrete
C204, Descriptive Nomenclature of Constituents of Natural Mineral Aggregates
C295, Petrographic Examintion of Aggregates for Concrete
C457, Microscopical Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete
C666, Resistance of Concrete to Rapid Freezing and Thawing
C671, Critical Dilation of Concrete Specimens Subjected to Freezing
C823, Recommended Practice for Examination and Sampling of Hardened Concrete in Constructions
C856, Recommended Practice for Petrographic Examination of Hardened Concrete

6.5.2 — *Stanton Walker Lecture by R. C. Mielenz, Diagnosing Concrete Failures,* November 18, 1964

Chapter 7 — Evaluation and recommendations

7.1 — Origin of distress

When evidences of structural distress or deterioration (chemical or mechanical) are uncovered, the cause of such conditions must be established. This information is necessary in order to evaluate the stability of the structure, estimate the length of service life remaining, select the best type of repair, and avoid a repetition of circumstances which lead to the currently existing condition.[27,28]

7.1.1 *Temperature and shrinkage cracks* — Cracks of this type are characterized by their fineness and absence of any indication of movement. They are usually shallow, a few inches in depth, and are not detected by sonic procedures. Where reinforcing steel exists near the surface, the cracks provide an access for water which may result in the formation of rust and subsequent discoloration or spalling. Steep temperature gradients during construction were likely responsible for excessive tensile strains at the surface. Drying during and subsequent to the curing period would have produced the same result. The shrinkage crack pattern is typically orthogonal or blocky. This surface cracking should not be confused with thermally induced deeper cracking occurring when dimensional change is restrained in newly placed concrete by rigid foundations or old lifts of concrete. Because all of the cracking described in this section likely is the result of construction conditions, the basic cause cannot be eliminated.

7.1.2 *Structural cracking* — Causes of this type of cracking are either excessive stress (which may be due to loading or stress pattern different from that expected by the designer) or inadequate concrete strength. The validity of the first possibility may be established by a review of the original design computations or a reanalysis of the structure design. Cracks originating from structural action will usually be substantial in width, and the opening may tend to increase as a result of continuous loading and creep of the concrete. Laboratory testing of cores or in-situ testing should reveal any deficiencies in concrete strength or unusual elastic modulus. These results should be compared with reliable and adequate construction records if available.

7.1.3 *Cavitation and erosion* — Cavitation distress of concrete surfaces can be very severe at high water velocities but can occur at low water velocities. The process of cavitation is associated with the creation and sudden collapse of negative pressures, resulting in the extraction of solid pieces of aggregate or mortar. Abrupt projections, uneven surfaces, and changes in direction of flow can cause cavitation conditions to develop.

Erosion is caused by suspended solids, generally fine and hard, which wear away the relatively soft cement paste or mortar. Characteristics of erosion damage are sharp ridges remaining on the harder portions of the exposed materials.

Abrasion is the result of large and hard bodies, such as aggregate cobbles or reinforcing steel, being entrapped and churned around on a relatively small concrete surface area. With time, these materials will wear away the concrete to form a hole, and the abrading action will continue until the cavity extends completely through the concrete mass.

7.1.4 *Cement-aggregate reaction* — Both the alkali-silica and alkali-carbonate reactions are characterized by reaction rims surrounding individual pieces of aggregate.[17,18] The effect in either instance is an expansion of the concrete due to the increased volume of the reaction products. The intensity and magnitude of such reactions will depend upon the mineralogical composition of the aggregate, the alkali content of the cementing material, availability of moisture, and the age of the structure. Only a very approximate estimate can be made of the rate of future expansion and the length of satisfactory service life remaining. Certain maintenance procedures have been effective, to a limited extent, in slowing the expansion and regression of concrete strength and elastic properties. Filling of cracks with grout or other suitable sealants and waterproofing exposed surfaces generally inhibits the entrance of moisture required in the reaction process. In some instances it may be necessary to provide additional structural support.

7.1.5 *Environmental distress* — Aggressive chemicals in soils or water, above various minimum concentrations, may be evidenced by discoloration around pattern cracking, disintegration of the mortar, or excessive expansion. The most common cause is likely a sodium, calcium, or magnesium sulfate occurring in the soil, in rivers, and in salt water.[19] Acidic waters will affect the concrete to lesser degree, but may cause corrosion of reinforcement not adequately covered by good quality concrete. The effects of many acids, salts, and other materials are described in Reference 11.

Leaching of lime from an inundated concrete surface such as the upstream face of a dam can result in up to 50 percent loss in strength.[21] Generally, only depths less than ¼ in. (6 mm) are affected. The leaching potential increases with increases in purity of the water and decrease in temperature. Lime has the peculiar property of being more soluble in cold water than warm water.

Virtually all mass concrete placed in recent years has included entrained air. While this has substantially reduced deterioration due to freeze-thaw actions, such distress still can occur under some circumstances. Inadequate air content, or an aggregate which is itself vulnerable to freeze-thaw deterioration, coupled with near-complete saturation, are examples of such conditions. Closely spaced, fine, parallel cracks near edges or joints may indicate that freeze-thaw expansions are occurring. Entrance of water into the cracks and subsequent freezing, further aggravates the condition.

7.1.6 *Physical and thermal properties* — Structural analyses of existing structures, either to determine stress magnitude and direction or to establish stability of the entire structure, require definite values of tensile strength, compressive strength, and elastic modulus. These data can be developed most reliably from drilled cores taken from the structure. When the structural analysis will require a knowledge of creep, the related parameters can likely be estimated from existing literature.[20] Similarly, the coefficient of thermal expansion (with consideration of aggregate type and moisture conditions) and Poisson's ratio may be estimated. If necessary, these properties can also be determined by tests on cores.

7.2 — Repair and rehabilitation

The objective of the recommendations is to present optimum alternatives for arresting deterioration, restoring deficient concrete, preventing leakage, and methods for reestablishing structural stability where such is deemed necessary by the structural engineer.

7.2.1 *Estimated service life* — The rate at which the surface concrete is deteriorating or disintegrating should permit making an estimate of the useful life of the structure, assuming no repairs will be made and continued exposure to the cause of the distress.

7.2.2 *Eliminating the cause* — Where the cause of deterioration can realistically be controlled, as for example eliminating the use or presence of aggressive chemicals, such practices should be identified and the potential benefits, in terms of extended service life and maintenance, presented. Where natural causes, such as sulfate soils, river water contamination, or freeze-thaw conditions are responsible, this should be so indicated.

7.2.3 *Surface protection* — Thin surface coatings are effective only in mildly distressed circumstances. Overlays of several inches thickness require removal of all concrete of doubtful quality, and replacement by a superior material. In-place polymerized concrete or mortar, epoxy mortar, fiber-reinforced concrete, or very low water-cement ratio concrete are alternative materials potentially capable of resisting mechanical abrasion or ingress of chemicals or water.[30,33,34,35]

7.2.4 *Restoring structural integrity* — Obvious indications of doubtful structural stability are cracks of substantial width, cracks which change in width with load changes or temperature cycles, or significant leakage. If the crack movement and the hydrostatic head is not high, leakage can be eliminated by routing out the crack, and injecting an elastomeric filler or a rigid epoxy mortar depending upon the probability of crack movement. In cases of high hydrostatic pressure, leakage may have to be controlled by drainage systems. When structural analyses indicate a fundamental deficiency in stability, post-tensioning between structural components or components and foundation rock should be considered. An adequate cover of grout or mortar around the steel strands is a necessity to avoid corrosion of the steel strands.

Chapter 8 — Report

8.1 — General

A formal report describing the condition of the concrete in the various structures of the project should be submitted to the owner or regulatory agency or Engineering organization requesting the evaluation. Hazardous conditions found during the evaluation should be reported to appropriate operating officials of the project without delay prior to preparation of the formal report.

8.2 — Contents of report

8.2.1 *Description of the project* — Regional vicinity maps for the project, plans, elevations, sections of the structures, and geologic maps when applicable should be shown. General purpose and operating requirements of the project and safety hazards and economic impacts involved in case of structural failure should be described.

8.2.2 *Pertinent design criteria for structures of project* — Significant structural design criteria upon which evaluation of the concrete was made and analyses, test methods, data and investigations pertinent to the evaluation should be described.

8.2.3 *Summary of data collected*

8.2.3.1 Existing records

8.2.3.2 Visual inspection of concrete

8.2.3.3 Analysis of existing instrumentation, investigations, inspections and test records
8.2.3.4 Results and analyses of new investigations and test data

8.3 — Evaluation of concrete in structures

The report should give an evaluation of the adequacy of the concrete based on current design and service conditions. If appropriate, recommendations for repair and maintenance required to assure future longevity and serviceability of the structures of the project should be given.

Chapter 9 — Information sources

9.1 — ASTM sources

The standards of the American Society for Testing and Materials applicable to testing and analysis referred to in this report are listed below with their serial designation including the year of adoption or revision. The standards listed were the latest editions at the time this report was prepared. Since these standards are revised frequently, generally in minor details only, the user of this report should check directly with the sponsoring group if it is desired to refer to the latest edition.

9.2 — ASTM standards

C33-77 Concrete Aggregates
C42-68 Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C85-66 Test for Cement Content of Hardened Portland Cement Concrete
C215-60 Test for Fundamental Transverse, Longitudinal and Torsional Frequencies of Concrete Specimens
C294-69 Descriptive Nomenclature of Constituents of Natural Mineral Aggregates
C295-65 Recommended Practice for Petrographic Examination of Aggregates for Concrete
C457-71 Recommended Practice for Microscopic Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete
C469-65 Test for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression
C597-71 Test for Pulse Velocity Through Concrete
C666-77 Test for Resistance of Concrete to Rapid Freezing and Thawing
C671-77 Test for Critical Dilation of Concrete Specimens Subjected to Freezing
C803-75T Test for Penetration Resistance of Hardened Concrete
C805-75T Test for Rebound Number of Hardened Concrete
C823-75 Recommended Practice for Examination and Sampling of Hardened Concrete in Constructions
C856-75 Recommended Practice for Petrographic Examination of Hardened Concrete

9.3 — References

1. *Inspection, Maintenance and Rehabilitation of Old Dams* (Engineering Foundation Conference, Pacific Grove, Sept. 1973), American Society of Civil Engineers, New York, 1974, 946 pp.

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This report was submitted to letter ballot of the committee which consisted of 21 members; 16 members returned affirmative ballots, 5 ballots were not returned.

ACI COMMITTEE 207

Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions

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ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction, and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be incorporated directly into the Project Documents.

ACI 210R-55
(Reaffirmed 1979)

Erosion Resistance of Concrete in Hydraulic Structures*

Reported by ACI Committee 210

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SYNOPSIS

Attention is given mainly to the physical erosion of concrete in hydraulic structures resulting from particles carried by flowing water and from pitting resulting from cavities forming and collapsing in water flowing at high velocities. Disintegration of concrete by chemical attack as may occur in hydraulic structures is also discussed.

Materials, mix proportions, and construction procedures which will make concrete more resistant to erosion are presented. Means of improving concrete resistance to chemical disintegration are also discussed.

CONTENTS

INTRODUCTION

This report is concerned mainly with the physical erosion of concrete in hydraulic structures resulting from particles carried by flowing water and from pitting resulting from cavities forming and collapsing in water flowing at high velocities. Disintegration of concrete by chemical attack as may occur in hydraulic structures is also discussed. Other types of disintegration are beyond the scope of this report.

In general, erosion resistance of concrete increases as the strength of the concrete is increased. Materials and methods which tend to increase the strength of concrete at the surface, or throughout the mass, also increase erosion resistance. However, the best concrete that can be made will not withstand the forces of cavitation or severe abrasion for a prolonged period. Materials, mix proportions, and construction procedures which will make concrete more resistant to erosion are discussed. Means of improving concrete resistance to chemical disintegration are also discussed.

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This report in form and substance as here submitted was approved unanimously by the committee as listed above.

†Deceased.

EROSION BY CAVITATION

Vapor bubbles will form in running water whenever the pressure at a point in the liquid is reduced to its vapor pressure in accordance with the following relationship:

$$P - \frac{V^2}{2g} C = V_p$$

where

P = apparent absolute pressure, ft of water
$V^2/2g$ = velocity head, ft
C = coefficient depending upon the shape and/or roughness of the surface boundary
V_p = vapor pressure of water at given temperature, ft of water, absolute

These vapor bubbles flow downstream with the water and upon entering an area of higher pressure collapse with great impact. When water vapor is compressed, the pressure of the vapor will increase until the vapor becomes saturated; then suddenly with very small increase in pressure the vapor will condense into a liquid state. The liquid occupies much less space than the vapor from which it was condensed, thus leaving a cavity. The collapse of these cavities has been termed "implosions," the opposite of "explosions," but is similar in effect. Repeated collapse of such cavities on or near the surface of the concrete will cause pitting. Pitting due to cavitation is readily recognized from the holes or pits formed, which are distinguished from the smoother worn-appearing surface caused by sand, silt, or rocks carried by the flowing water.

Boundary irregularities and shape cause local reductions in pressure when water flows past them at high velocity. When the size and nature of the irregularity and the flow velocity are such that reduced pressure is equal to the vapor pressure of water, cavitation will occur. Damage from cavitation is not common in open conduits at water velocities below 40 ft per sec. Fig. 1

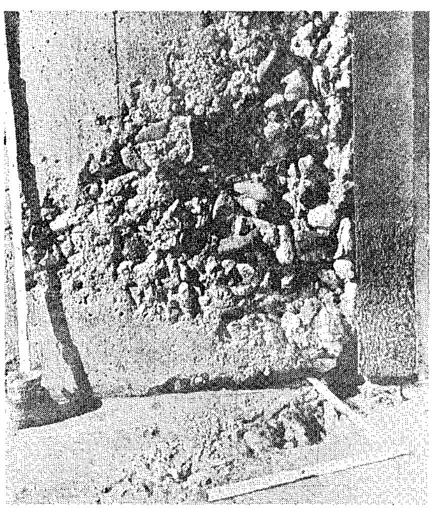

Fig. 1—Pitting in Parker Dam gate pier below gate slot. Note pitting in metal of gate guide. This type of pitting is common and has occurred near the control gates on other structures. Maximum velocity was 50 ft per sec

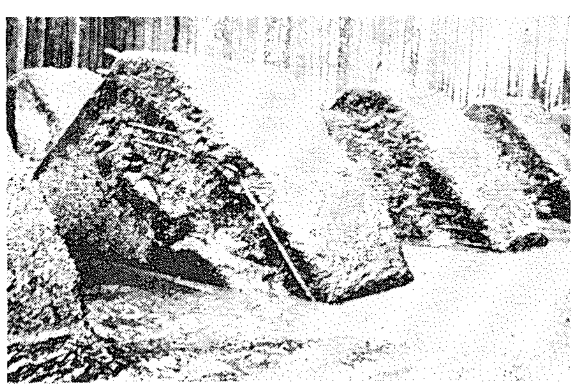

Fig. 2—Pitting in concrete baffles in the Bonneville Dam stilling basin after 17 years of heavy service. The arrow indicates the direction of the high velocity water during each seasonal flood. The attrition on baffles was progressive and had reached a point, when they were repaired in 1955, where hydraulic conditions in the stilling basin were adversely affected by reduction in the efficiency of the baffles

shows the damage which resulted at Parker Dam for a maximum velocity of about 50 ft per sec. Fig. 2 shows damage to concrete baffles in the Bonneville Dam stilling basin after 17 years of service. Velocities ranged from 60 to 70 ft per sec. Concrete in closed conduits has been pitted by cavitation at velocities as low as 25 ft per sec where the air pressure was reduced by the sweep of the flowing water. Fig. 3 shows such damage in a conduit below a control gate, produced at a velocity of about 25 ft per sec. At higher velocities the forces of cavitation are sufficient to erode away large quantities of high-quality concrete and to penetrate through thick steel plates in a comparatively short time. Concrete in spillways and outlet works of many high dams has been severely damaged by forces of cavitation. In closed conduits the liquid can be made more compressible by introducing air into the flowing water near the point of disturbance, and the forces resulting from collapsing vapor bubbles can be reduced.

The best means, however, of protecting concrete from forces of cavitation is the elimination of these forces, whenever possible, by design and construction procedures which will produce smooth, uniform flow in the hydraulic structure. Abrupt changes in slope and curvature, particularly adverse changes which tend to allow the flow to pull away from the concrete surface, should be avoided, and care should be exercised to obtain a smooth surface free from irregularities. An example of damage, on the inclined face of Grand Coulee Dam, resulting from poor alignment below a place where the forms had sprung out of line is shown in Fig. 4. There are a number of examples where the bottom of inclined tunnels, spillway buckets, stilling pools, and piers and crests just downstream from gate guide slots have been damaged by forces of cavitation, and these locations need special attention in design and construction. Computed cavitation-free designs are often so conservative that their costs are prohibitive. In such cases hydraulic model studies may be used to arrive at acceptable designs with due regard to cost.

The sloping faces of overflow spillways have not been damaged by cavitation where reasonable care has been taken to produce surfaces of sound concrete without irregularities. Small voids or "bug holes" which occur on formed surfaces are not necessarily objectionable, but obstructions which protrude above the plane of the surface will result in pitting downstream from the obstruction.

Concrete is comparatively strong in compression and there are many tests and experiences which show that dense concrete can withstand the impact of a jet of water hitting it at velocities as high as 100 ft per sec without damage at the point of impact. Damage has occurred in some cases as a result of water impact, and it is indicated that such failures may be due to the pressures built up in the pores of the concrete which are sufficient to cause failure in the concrete in tension at points of reduced pressure. For this reason it is desirable to use dense concrete having an impermeable surface when it will be expected to withstand the impact of water flowing at high velocities.

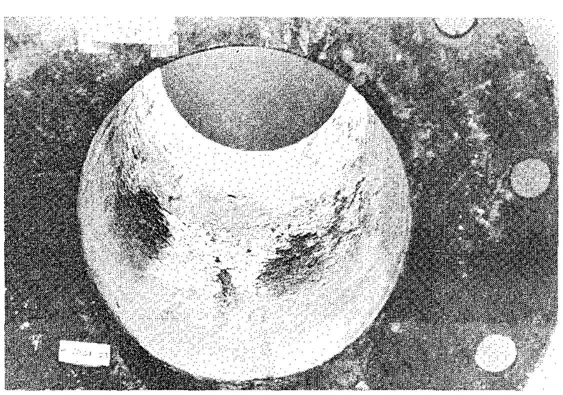

Fig. 3—Pitting in 6-in. ID mortar lined pipe below control gate–velocity about 25 ft per sec

Fig. 4—View looking up spillway face of Grand Coulee Dam showing pitted area at elevation 990 below place where form sprung out of line during construction

EROSION BY ABRASION

Erosion of concrete by silt, sand, gravel, and other solids can be as severe as that caused by cavitation. Fig. 5 shows erosion caused by movement of sand and gravel by eddy currents in the bucket of Grand Coulee Dam. Stilling basins which are not self-cleaning in which rocks and sand collect are eroded by the movement of solids by eddy currents in the pool, and concrete over which large quantities of sand and gravel are transported by floods may be seriously eroded.

Concrete in the invert of the 20-ft diameter, 1300 ft long tunnel at Anderson Ranch Dam was worn away to a depth of about 3 in. while it was used for diverting the flow of the river for 43 months during construction of the dam.

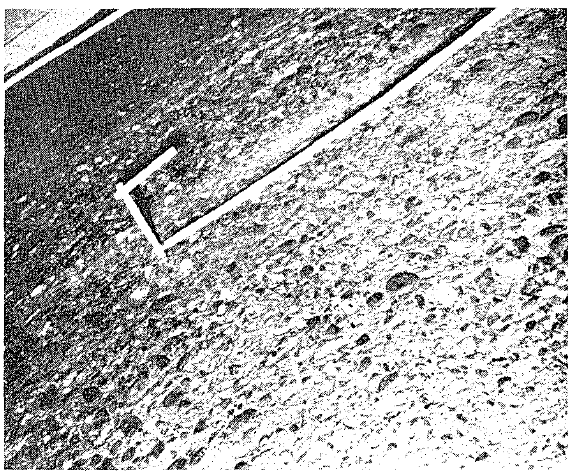

Fig. 5 — Surface of de-watered bucket of Grand Coulee Dam showing condition of surface eroded by sand and gravel

The water carried large quantities of silt, sand, and gravel during the spring runoff, and when the tunnel was unwatered the invert was covered to a depth of several feet with such material. The wear was fairly uniform on all types of aggregate and the exposed surfaces of the larger aggregate were smooth and flat. Some of the 1:2 dry-packed mortar patches in this tunnel were completely eroded away, and in general the mortar patches were eroded more than the surrounding concrete. Maximum velocity of the water in the tunnel was about 30 ft per sec. The new low-slump concrete which has been installed in this tunnel and subjected to high velocities of relatively clear water since it was converted to an outlet tunnel, shows only slight wear. Similar erosion was experienced in the diversion tunnels of Hoover Dam prior to their conversion to outlet and spillway tunnels. There are many cases where the concrete of dams and tunnel linings has been damaged by erosion during the construction period, and this possibility should not be overlooked in design considerations.

Fig. 6* shows that the comparatively low bottom velocity of 5 ft per sec is capable of moving rock particles as large as 4 in. in diameter. Apparently, the rate of erosion is dependent on the quantity, shape, size, and hardness of the particles being transported, the velocity of the water, and the quality of the concrete. Concrete-lined irrigation canals which usually carry small quantities of solids show no appreciable erosion after years of service for velocities as high as 6 ft per sec.

*From "The Start of Bed-Load Movement and the Relation between Competent Bottom Velocities in a Channel and the Transportable Sediment Size," by Norendra Kumar Berry, University of Colorado, 1948.

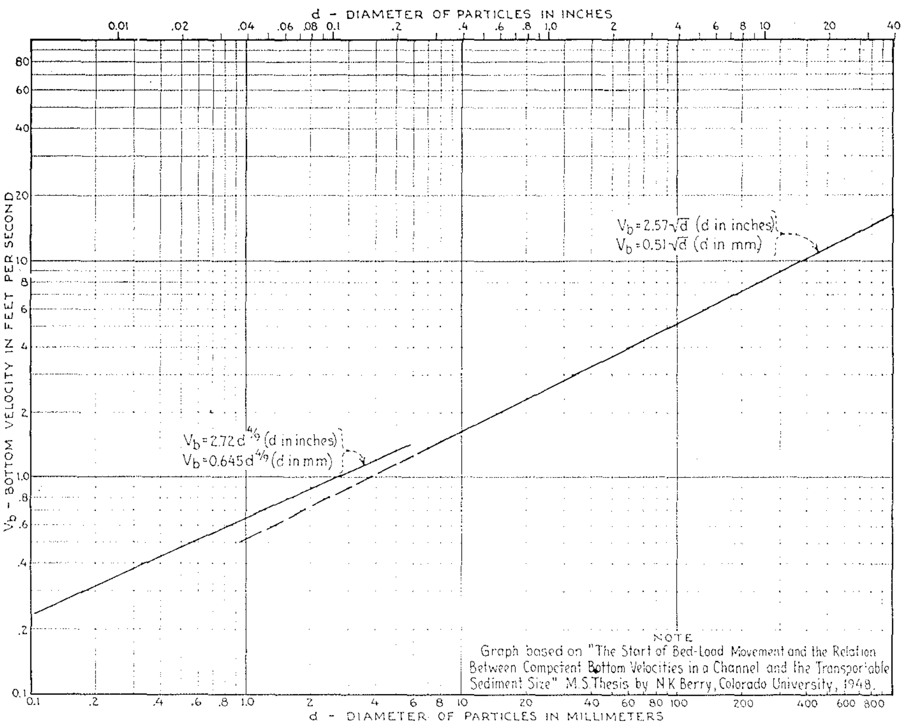

Fig. 6—Relation between competent bottom velocity and transportable sediment size

Where it is expected that the conduit will carry solids or that abrasion will result from solids and eddy currents, the concrete should be of the highest quality, because abrasion resistance increases as strength of the concrete is increased. It is not necessary to be as particular about alignment and surface smoothness where only abrasion is expected and where the velocity of the water will not exceed 40 ft per sec.

EROSION BY CHEMICAL ATTACK

The compounds present in set portland cement are attacked by water and many salt and acid solutions, though fortunately in most instances the action on an impermeable mass of hardened portland cement paste is so slow as to be unimportant. There are, however, certain conditions to which concretes may be exposed producing reactions which become serious.

One of the compounds formed when cement and water combine is hydrated lime, which is readily dissolved by water and more aggressively dissolved by pure mineral-free water as is found in some mountain streams. Dissolved carbon dioxide is contained in some fresh waters in sufficient quantity to make the water slightly acid and add to its aggressiveness. Serious attacks by fresh water, partly on concrete surfaces, but more markedly in defective (porous or cracked) parts of interior concrete of conduits have been reported in Scandinavian countries. In America there are many instances where the surface of the concrete has been etched by fresh water flowing over it, but serious damage from this cause is uncommon. A review of raw water in many reservoirs throughout the United States, published in V. 3 of the *Transactions of the Fourth Congress on Large Dams*, indicates a nearly neutral condition for most of these waters.

The main concrete corrosion problem in a sewer is chemical attack by sulfuric acid in the crown of the sewer. Sanitary sewage (domestic house sewage derived principally from dwellings, business buildings, institutions, and the like, which may or may not contain some groundwater, surface water, storm water, and a small proportion of industrial waste liquid) generally fluctuates slightly above and below a pH of 7.0, averaging out as neutral. Such sewage does not attack concrete. However, conditions of temperature, sewage concentration, and velocity of flow may be such that sulfur-bearing materials in sewage are converted to hydrogen sulfide gas by bacteria reposing in the silt or scum in the sewer. If sewage flows very slowly or is stagnant, the rate at which hydrogen sulfide is generated may exceed the rate at which it can be oxidized by the oxygen dissolved in the sewage and hydrogen sulfide gas may emerge into the atmosphere above the sewage. Before hydrogen sulfide gas can do any damage to the sewer crown it must come in contact with moisture on the crown where, under certain conditions, it is converted to sulfuric acid. Although information is available which should enable the engineer to design, construct, and operate a sewer so that the development of sulfuric acid is practically eliminated, it is not always possible to accomplish this, and corrosion of concrete sewer pipe from this acid is a serious problem in some cities.

No portland cement concrete, regardless of what may be its other ingredients, will long withstand water of high acid concentration. Where strong acid corrosion is indicated, an appropriate surface covering or treatment should be used. Replacement of a portion of the portland cement by a suitable pozzolan in amounts up to 30 percent will improve the resistance of concrete to weak acid attack.

Sulfates of sodium, magnesium, and calcium frequently encountered in the "alkali" soils and groundwaters of the western United States attack concrete aggressively. The sulfates react chemically with the hydrated lime and hydrated calcium aluminate in the cement paste to form calcium sulfate and calcium sulfoaluminate, which is accompanied by considerable expansion and disruption of the concrete. Concrete containing cement low in tricalcium aluminate (C_3A) is highly resistant to attack by sulfate-laden soils and waters. Whenever the sulfate (as SO_4) in the water is above 1000 ppm, a sulfate-resisting cement low in C_3A should be used. Use of a suitable pozzolan for replacement of Type I or Type II cements (which are usually relatively high in C_3A) in amounts approximating 30 percent will improve the sulfate resistance of the concrete. Rich mixes are more resistant to sulfate attack than lean ones.

MIX PROPORTIONS AND MATERIALS

It has been conclusively demonstrated that resistance of concrete to forces of cavitation and abrasion increases with increased compressive strength of the concrete. It is recommended that when the structure will be subjected to forces of cavitation or abrasion the mix be proportioned for a strength of at least 6000 psi. Where sulfate attack is expected, the water-cement ratio of the concrete should be held below 0.45 by weight. The ACI Committee 613 report* gives information on mix proportions, and the report of ACI Committee 614† gives information on measuring, mixing, and placing of concrete.

Cement

The type of portland cement is of little importance from an abrasion and cavitation-resistance standpoint, provided adequate compressive strength is obtained at the desired time. Cements low in tricalcium aluminate (such as Type V) are much more resistant to sulfate attack than cements high in this compound. No portland cement is resistant to acid attack.

Pozzolans

Pozzolans are siliceous materials, natural or artificial, which, though not cementitious in themselves, contain constituents that will, at ordinary temperatures, combine with lime in the presence of water to form compounds which have low solubility and possess cementing properties. Suitable pozzolans, when used to replace a portion of the portland cement in the concrete, com-

*ACI Committee 613, "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)," ACI JOURNAL, Sept. 1954, *Proc.* V. 51, pp. 49-64.
†ACI Committee 614, "Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)," ACI JOURNAL, June 1945, *Proc.* V. 51, pp. 625-650.

bine with the lime liberated during the hydration processes to form a more stable product. Usually concrete is made more impermeable through addition of a finely divided pozzolan. Pozzolans improve the resistance of concrete to leaching and weak acid attack and to sulfate attack where they replace portions of cements relatively high in tricalcium aluminate. Concrete in which a portion of the cement has been replaced by a pozzolan usually develops strength slower than a concrete made with straight portland cement and this should not be overlooked where abrasion is expected soon after placing the concrete.

Aggregates

It has been demonstrated that the larger particles of aggregate are plucked out or pushed out of the concrete by the forces of cavitation more easily than the smaller particles. It is indicated, therefore, that the maximum size aggregate should be limited to ¾ in. where cavitation might occur. The aggregate should be sound, but its hardness is not of as great importance as in the case of abrasion where the aggregates resist abrasion after the layer of mortar has been eroded away. Good bond is more important than hardness in the case of cavitation.

In the case of abrasion, the aggregates are not pulled out of the matrix, but are worn down by solids carried in the flowing water. An aggregate which will resist wear is desirable in this instance, and the fine and coarse aggregate should preferably contain not more than 2 percent of soft particles. Aggregates are more resistant to chemical attack than cement paste, and any aggregate which meets the usual specifications should be suitable where this is the main consideration.

Entrained air

Entrained air, obtained either by use of an air-entraining cement or an admixture, greatly improves the workability of concrete and its resistance to weathering. For the same cement content the strength of richer mixes is reduced slightly by air entrainment. But, in spite of this reduction in strength, use of entrained air in the proportions listed in "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)," is recommended wherever freezing and thawing is expected. Entrained air also improves the resistance of concrete to sulfate and acid attack under some exposure conditions, due apparently to improved impermeability.

Slump

Concrete for structures should be as dry as can be adequately placed, but in no case should the slump be over 3 in. Dry-tamped concrete used in machine-manufactured pipe should be on the moist side for best results.

FINISHES AND FINISHING

The formula given previously in this report shows that as the depth of water over the surface is increased the tendency toward formation of cavities in the water is decreased. Nevertheless, where water velocities of more than 40 ft per sec are expected, the concrete should be built to accurate alignment and evenness of surface. Abrupt irregularities caused by displaced or misplaced form sheathing, lining or form sections, or by loose knots in forms or otherwise defective form lumber, should not exceed ¼ in. parallel to the direction of flow and ⅛ in. at right angles to the direction of flow. Gradual irregularities which can be measured by a 5 ft long template should not exceed ¼ in. For formed surfaces of open, overflow-type spillways, irregularities may be increased to twice the size of those listed. For velocities above 100 ft per sec, irregularities should be reduced to half the size of those listed above.

Forms may be made of steel, wood, aluminum, or any other material which will give the desired smoothness and alignment. Green lumber should not be used because, in addition to shrinkage and warping which may occur, the tannic acid in the lumber may soften the surface of the concrete.

Forms should be removed at the earliest practicable time so that curing may proceed without delay and necessary repairs or surface treatment done while the concrete is still green and conditions are most favorable for good bond. The ACI Committee 604 report* gives information on form removal during cold weather. Offsets and bulges should be removed by grinding to specification limits.

The concrete of unformed surfaces should contain just sufficient mortar to avoid the necessity for excessive floating. If the mix is wet and oversanded, an excess of water and fine material will be brought to the surface and the result will be a layer of inferior mortar having a high water-cement ratio with a tendency to dust, craze, crack, and possibly separate from the mass beneath. Working of the surface in the various finishing operations should be the minimum necessary to produce the desired finish. The first step in the finishing operation is the leveling and screeding of the concrete to produce an even and uniform surface. This is followed by floating, which should not be started until some stiffening has taken place in the surface of the concrete and the moisture film or "shine" has disappeared. The floating should work the concrete no more than necessary to produce a surface that is uniform in texture and free of screed marks.

From an erosion standpoint, floated surfaces are of sufficient smoothness for velocities less than 40 ft per sec. For high velocities the surface should be smoothed by steel troweling. Steel troweling should not be started until after the moisture film and "shine" have disappeared from the floated surface and after the concrete has hardened enough to prevent an excess of fine material and water from being worked to the surface. Steel troweling should be performed with firm pressure such as will flatten the sandy texture of the floated surface and produce a dense, uniform surface. Dense, impermeable surfaces are more resistant to impact by high-velocity jets than porous ones. Through use of absorptive forms or the vacuum process on flat surfaces, the denseness and strength of the surface of concrete can be increased above that which can usually be obtained by proper placing and finishing methods. These processes remove water from the surface of the concrete while it is still in the plastic state. Concrete surfaces can be protected from forces of cavitation and abrasion by rubber-like coatings (see "Resistance of Concrete and Protective Coatings to Forces of Cavitation," by W. H. Price and G. B. Wallace, ACI JOURNAL, Oct. 1949, *Proc.* V. 46, pp. 109-120). Tests have also shown that metallic aggregate greatly improves the resistance of concrete surfaces to erosion and pitting as a result of cavitation.

Precast concrete pipe is manufactured in a number of different ways, such as by placing and vibrating concrete in vertical forms as in any wall, by tamping and packing relatively dry concrete as in the packerhead and tamped processes, by spinning plastic mortar or concrete in a horizontal form, and by spinning, vibrating, and rolling relatively dry concrete in a rotating horizontal form. Conduits are also lined by various processes of applying and smoothing mortar on the inside surface of the conduit. There is considerable difference in density and strength of the surface concrete produced under the different methods and by different manufacturers under the same method due to variations in materials, consistency, and curing employed. It is recommended, therefore, after it has been decided what surface is desirable for the conditions of the job, that a process and procedure which will obtain this surface be specified. Specifications of the American Society for Testing Materials and the American Water Works Assn. for concrete pipe and mortar-lined steel pipe are given in the bibliography.

CURING

Proper hydration of the cement to form hard and durable concrete requires that the concrete be maintained in a moist condition for a suitable period,

*ACI Committee 604, "Proposed Recommended Practice for Winter Concreting," ACI JOURNAL, Oct. 1955, *Proc.* V. 52, pp. 113-130.

usually 14 days. Usual procedure for accomplishing this is to keep the exposed surface continuously (not periodically) wet by spraying, ponding, or by covering with earth, sand, or burlap maintained in a wet condition. Moist curing should start as soon as forms can be loosened or removed. Curing of unformed concrete should start immediately after the concrete has taken its initial set. Under certain conditions it is desirable to cure concrete by applying to the exposed surfaces a sealing compound designed to restrict evaporation of the mixing water. An effective compound, properly applied and maintained for at least 28 days, will, under most conditions, retain enough moisture for adequate curing. A white-pigmented compound has been found to be especially suitable for hot weather curing, as it considerably decreases the heat which would be absorbed from direct sunlight.

Although attention to curing requirements is important at all times, it is especially so in hot dry weather because of the greater danger of crazing and cracking. Concrete as deposited should have a temperature of not more than 90 F because it has been shown that strength and durability of concrete decreases as the placing temperature is raised. Concrete placed in cold weather should be protected from freezing in accordance with the ACI Committee 604 report.

REPAIRS OF ERODED AREAS

Where concrete has been damaged by erosion it is almost certain that the repaired section will be damaged also unless the cause of the erosion is removed. The best concrete made will not withstand the forces of cavitation or severe abrasion for a prolonged period. It may be more economical, however, to replace the concrete periodically than to reshape the structure to produce streamlined flow or to eliminate the solids causing abrasion. Furthermore, it may be undesirable to streamline those portions of the structure designed to dissipate energy such as dentated sills and other types of obstructions to the flow of water placed in stilling basins of spillways. Such sills should be made of the best possible concrete.

In repairing concrete it is necessary that attention be paid to many details, because experience has demonstrated that repairs that are carelessly made later become defective and have to be replaced. All damaged concrete and loose and broken particles should be removed. Where the thickness of section permits, holes to be repaired should be enlarged to a minimum depth of 6 in. with a minimum area of ½ sq ft in reinforced and 1 sq ft in unreinforced concrete. There should be a clearance of at least 1 in. around each exposed reinforcing bar; they should not be left partially embedded. The top edge of the hole in walls at the face of the structure should be cut to a fairly horizontal line. If the shape of the eroded area makes it advisable, the top of the cut may be stepped down and continued on a horizontal line. In all cases in walls or floors it is desirable to make a square edge by cutting around the area to be repaired with a concrete saw prior to chipping out the area to be repaired.

It is extremely important that the concrete to be repaired be thoroughly cleaned by wet sandblasting, followed by washing with an air-water jet. The holes should be kept continuously wet for not less than 12 hr prior to placing new concrete. Cavities should be free of any water at the time of placing and preferably should be surface dry. The replacement concrete should have a slump as low as is consistent with thorough vibration and good placement.

Where velocities above 40 ft per sec are encountered, the repaired areas should be made smooth. It may be necessary to grind them after they have hardened.

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ACI Standard

Recommended Practice for Selecting Proportions for Normal and Heavyweight Concrete (ACI 211.1-77)*

Reported by ACI Committee 211

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Describes, with examples, two methods for selecting and adjusting proportions for normal weight concrete. One method is based on an estimated weight of the concrete per unit volume; the other is based on calculations of the absolute volume occupied by the concrete ingredients. The procedures take into consideration the requirements for placeability, consistency, strength, and durability. Example calculations are shown for both methods, including adjustments based on the characteristics of the first trial batch.

The proportioning of heavyweight concrete for such purposes as radiation shielding and bridge counterweight structures is described in an appendix. This appendix uses the absolute volume method which is generally accepted and is more convenient for heavyweight concrete.

Keywords: adsorption; aggregates; **air-entrained concretes;** air entrainment; cement content; coarse aggregates; concrete durability; **concretes;** consistency; durability; exposure; fine aggregates; heavyweight aggregates; **heavyweight concretes; mix proportioning;** quality control; radiation shielding; slump tests; volume; water-cement ratio; workability.

1. SCOPE

1.1—This recommended practice describes methods for selecting proportions for concrete made with aggregates of normal and high density (as distinguished from lightweight and special high density aggregates) and of workability suitable for usual cast-in-place construction (as distinguished from special mixtures for concrete products manufacture).

1.2—The methods provide a first approximation of proportions intended to be checked by trial batches in the laboratory or field and adjusted, as necessary, to produce the desired characteristics of the concrete.

1.3—U. S. customary units are used in the main body of the text. Adaptation for the metric system is provided in Appendix 1, and demonstrated in an example problem in Appendix 2.

1.4—Test methods mentioned in the text are listed in Appendix 3.

2. INTRODUCTION

2.1—Concrete is composed principally of cement, aggregates, and water. It will contain some amount of entrapped air and may also contain purposely entrained air obtained by use of an admixture or air-entraining cement. Admixtures are also frequently used for other purposes such as to accelerate, retard, improve workability, reduce mixing water requirement, increase strength, or alter other properties of the concrete.

2.2—The selection of concrete proportions involves a balance between reasonable economy and requirements for placeability, strength, durability, density, and appearance. The required characteristics are governed by the use to which the concrete will be put and by conditions ex-

*Adopted as a standard of the American Concrete Institute in September 1977, to supersede ACI 211.1-74 (Revised 1975), in accordance with the Institute's standardization procedure.

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pected to be encountered at the time of placement. These are often, but not always, reflected in specifications for the job.

2.3—The ability to tailor concrete properties to job needs reflects technological developments which have taken place, for the most part, since the early 1900s. The use of the water-cement ratio as a tool for estimating strength was recognized about 1918. The remarkable improvement in durability resulting from the entrainment of air was recognized in the early 1940s. These two significant developments in concrete technology have been augmented by extensive research and development in many related areas, including the use of admixtures to counteract possible deficiencies, develop special properties, or achieve economy.* It is beyond the scope of this discussion to review the theories of concrete proportioning which have provided the background and sound technical basis for the relatively simple methods of this recommended practice. More detailed information can be obtained from the list of references.

2.4—Proportions calculated by any method must always be considered subject to revision on the basis of experience with trial batches. Depending on circumstances, the trial mixes may be prepared in a laboratory or, perhaps preferably, as full-size field batches. The latter procedure, when feasible, avoids possible pitfalls of assuming that data from small batches mixed in a laboratory environment will predict performance under field conditions. Trial batch procedures and background testing are described in Appendix 3.

3. BASIC RELATIONSHIP

3.1—Concrete proportions must be selected to provide necessary placeability, strength, durability, and density for the particular application. Well established relationships governing these properties are discussed briefly below.

3.2—*Placeability* (including satisfactory finishing properties) encompasses traits loosely accumulated in the terms "workability" and "consistency." For the purpose of this discussion, *workability* is considered to be that property of concrete which determines its capacity to be placed and consolidated properly and to be finished without harmful segregation. It embodies such concepts as moldability, cohesiveness, and compactability. It is affected by the grading, particle shape and proportions of aggregate, the amount of cement, the presence of entrained air, admixtures, and the consistency of the mixture. Procedures in this recommended practice permit these factors to be taken into account to achieve satisfactory placeability economically.

3.3—*Consistency*, loosely defined, is the wetness of the concrete mixture. It is measured in terms of slump—the higher the slump the wetter the mixture—and it affects the ease with which the concrete will flow during placement. It is related to but not synonymous with workability. In properly proportioned concrete, the unit water content required to produce a given slump will depend on several factors. Water requirement increases as aggregates become more angular and rough textured (but this disadvantage may be offset by improvements in other characteristics such as bond to cement paste). Required mxing water decreases as the maximum size of well graded aggregate is increased. It also decreases with the entrainment of air. Mixing water requirement may often be significantly reduced by certain admixtures.

3.4—*Strength*. Strength is an important characteristic of concrete, but other characteristics such as durability, permeability, and wear resistance are often equally or more important. These may be related to strength in a general way but are also affected by factors not significantly associated with strength. For a given set of materials and conditions, concrete strength is determined by the net quantity of water used per unit quantity of cement. The net water content excludes water absorbed by the aggregates. Differences in strength for a given water-cement ratio may result from changes in: maximum size of aggregate; grading, surface texture, shape, strength, and stiffness of aggregate particles; differences in cement types and sources; air content; and the use of admixtures which affect the cement hydration process or develop cementitious properties themselves. To the extent that these effects are predictable in the general sense, they are taken into account in this recommended practice. However, in view of their number and complexity, it should be obvious that accurate predictions of strength must be based on trial batches or experience with the materials to be used.

3.5—*Durability*. Concrete must be able to endure those exposures which may deprive it of its serviceability—freezing and thawing, wetting and drying, heating and cooling, chemicals, deicing agents, and the like. Resistance to some of these may be enhanced by use of special ingredients: low-alkali cement, pozzolans, or selected aggregate to prevent harmful expansion due to the alkali-aggregate reaction which occurs in some areas when concrete is exposed in a moist environment: sulfate resisting cement or pozzolans for concrete exposed to seawater or sulfate-

*See ACI Committee 212, "Admixtures for Concrete," ACI JOURNAL, *Proceedings* V. 60, No. 11, Nov. 1963, pp. 1525-1534.

bearing soils; or aggregate free of excessive soft particles where resistance to surface abrasion is required. Use of a low water-cement ratio will prolong the life of concrete by reducing the penetration of aggressive liquids. Resistance to severe weathering, particularly freezing and thawing, and to salts used for ice removal is greatly improved by incorporation of a proper distribution of entrained air. Entrained air should be used in all exposed concrete in climates where freezing occurs.*

3.6—*Density.* For certain applications concrete may be used primarily for its weight characteristic. Examples of applications are counterweights on lift bridges, weights for sinking oil pipelines under water, shielding from radiation, and for insulation from sound. By using special aggregates, placeable concrete of densities as high as 350 lb per cu ft can be obtained—see Appendix 4.

4. BACKGROUND DATA

4.1—To the extent possible, selection of concrete proportions should be based on test data or experience with the materials actually to be used. Where such background is limited or not available, estimates given in this recommended practice may be employed.

4.2—The following information for available materials will be useful:

4.2.1 Sieve analyses of fine and coarse aggregates

4.2.2 Unit weight of coarse aggregate

4.2.3 Bulk specific gravities and absorptions of aggregates

4.2.4 Mixing water requirements of concrete developed from experience with available aggregates

4.2.5 Relationships between strength and water-cement ratio for available combinations of cement and aggregate

4.3—Estimates from Tables 5.3.3 and 5.3.4, respectively, may be used when the last two items of information are not available. As will be shown, proportions can be estimated without the knowledge of aggregate specific gravity and absorption, Item 4.2.3.

5. PROCEDURE

5.1—The procedure for selection of mix proportions given in this section is applicable to normal weight concrete. Although the same basic data and procedures can be used in proportioning heavyweight concrete, additional information as well as sample computations for this type of concrete are given in Appendix 4.

5.2—Estimating the required batch weights for the concrete involves a sequence of logical, straightforward steps which, in effect, fit the characteristics of the available materials into a mixture suitable for the work. The question of suitability is frequently not left to the individual selecting the proportions. The job specifications may dictate some or all of the following:

5.2.1 Maximum water-cement ratio

5.2.2 Minimum cement content

5.2.3 Air content

5.2.4 Slump

5.2.5 Maximum size of aggregate

5.2.6 Strength

5.2.7 Other requirements relating to such things as strength overdesign, admixtures, and special types of cement or aggregate.

5.3—Regardless of whether the concrete characteristics are prescribed by the specifications or are left to the individual selecting the proportions, establishment of batch weights per cubic yard of concrete can best be accomplished in the following sequence:

5.3.1 *Step 1. Choice of slump.* If slump is not specified, a value appropriate for the work can be selected from Table 5.3.1. The slump ranges shown apply when vibration is used to consolidate the concrete. Mixes of the stiffest consistency that can be placed efficiently should be used.

TABLE 5.3.1—RECOMMENDED SLUMPS FOR VARIOUS TYPES OF CONSTRUCTION

Types of construction	Slump, in.	
	Maximum*	Minimum
Reinforced foundation walls and footings	3	1
Plain footings, caissons, and substructure walls	3	1
Beams and reinforced walls	4	1
Building columns	4	1
Pavements and slabs	3	1
Mass concrete	2	1

*May be increased 1 in. for methods of consolidation other than vibration.

5.3.2 *Step 2. Choice of maximum size of aggregate.* Large maximum sizes of well graded aggregates have less voids than smaller sizes. Hence, concretes with the larger-sized aggregates require less mortar per unit volume of concrete. Generally, the maximum size of aggregate should be the largest that is economically available and consistent with dimensions of the structure. In no event should the maximum size exceed one-fifth of the narrowest dimension between sides of forms, one-third the depth of slabs, nor three-fourths of the minimum clear spacing between individual reinforcing bars, bundles of bars, or pretensioning strands. These limitations are sometimes waived if workability and methods of consolidation are such that the concrete can be

*For further details, see ACI Committee 201, "Durability of Concrete in Service," ACI JOURNAL, *Proceedings* V. 59, No. 12, Dec. 1962, pp. 1771-1820.

TABLE 5.3.3—APPROXIMATE MIXING WATER AND AIR CONTENT REQUIREMENTS FOR DIFFERENT SLUMPS AND NOMINAL MAXIMUM SIZES OF AGGREGATES*

Slump, in.	Water, lb per cu yd of concrete for indicated nominal maximum sizes of aggregate							
	3/8 in.	1/2 in.	3/4 in.	1 in.	1 1/2 in.	2 in.†	3 in.†	6 in.†
Non-air-entrained concrete								
1 to 2	350	335	315	300	275	260	240	210
3 to 4	385	365	340	325	300	285	265	230
6 to 7	410	385	360	340	315	300	285	—
Approximate amount of entrapped air in non-air-entrained concrete, percent	3	2.5	2	1.5	1	0.5	0.3	0.2
Air-entrained concrete								
1 to 2	305	295	280	270	250	240	225	200
3 to 4	340	325	305	295	275	265	250	220
6 to 7	365	345	325	310	290	280	270	—
Recommended average‡ total air content, percent, for level of exposure:								
Mild exposure	4.5	4.0	3.5	3.0	2.5	2.0	1.5§**	1.0§**
Moderate exposure	6.0	5.5	5.0	4.5	4.5	4.0	3.5§**	3.0§**
Extreme exposure††	7.5	7.0	6.0	6.0	5.5	5.0	4.5§	4.0§

*These quantities of mixing water are for use in computing cement factors for trial batches. They are maxima for reasonably well-shaped angular coarse aggregates graded within limits of accepted specifications.

†The slump values for concrete containing aggregate larger than 1½ in. are based on slump tests made after removal of particles larger than 1½ in. by wet-screening.

‡Additional recommendations for air content and necessary tolerances on air content for control in the field are given in a number of ACI documents, including ACI 201, 345, 318, 301, and 302. ASTM C 94 for ready-mixed concrete also gives air content limits. The requirements in other documents may not always agree exactly, so in proportioning concrete consideration must be given to selecting an air content that will meet the needs of the job and also meet the applicable specifications.

§For concrete containing large aggregates which will be wet-screened over the 1½ in. sieve prior to testing for air content, the percentage of air expected in the 1½ in. minus material should be as tabulated in the 1½ in. column. However, initial proportioning calculations should include the air content as a percent of the whole.

**When using large aggregate in low cement factor concrete, air entrainment need not be detrimental to strength. In most cases mixing water requirement is reduced sufficiently to improve the water-cement ratio and to thus compensate for the strength reducing effect of entrained air on concrete. Generally, therefore, for these large maximum sizes of aggregate, air contents recommended for extreme exposure should be considered even though there may be little or no exposure to moisture and freezing.

††These values are based on the criteria that 9 percent air is needed in the mortar phase of the concrete. If the mortar volume will be substantially different from that determined in this recommended practice, it may be desirable to calculate the needed air content by taking 9 percent of the actual mortar volume.

placed without honeycomb or void. When high strength concrete is desired, best results may be obtained with reduced maximum sizes of aggregate since these produce higher strengths at a given water-cement ratio.

5.3.3 *Step 3. Estimation of mixing water and air content.* The quantity of water per unit volume of concrete required to produce a given slump is dependent on the maximum size, particle shape and grading of the aggregates, and on the amount of entrained air. It is not greatly affected by the quantity of cement. Table 5.3.3 provides estimates of required mixing water for concretes made with various maximum sizes of aggregate, with and without air entrainment. Depending on aggregate texture and shape, mixing water requirements may be somewhat above or below the tabulated values, but they are sufficiently accurate for the first estimate. Such differences in water demand are not necessarily reflected in strength since other compensating factors may be involved. For example, a rounded and an angular coarse aggregate, both well and similarly graded and of good quality, can be expected to produce concrete of about the same compressive strength for the same cement factor in spite of differences in water-cement ratio resulting from the different mixing water requirements. Particle shape per se is not an indicator that an aggregate will be either above or below average in its strength-producing capacity.

Table 5.3.3 indicates the approximate amount of entrapped air to be expected in non-air-entrained concrete in the upper part of the table and shows the recommended average air content for air-entrained concrete in the lower part of the table. If air entrainment is needed or desired, three levels of air content are given for each aggregate size depending on the purpose of the entrained air and the severity of exposure if entrained air is needed for durability:

Mild exposure—When air entrainment is desired for a beneficial effect other than durability, such as to improve workability or cohesion or in low cement factor concrete to improve strength, air contents lower than those needed for durability can be used. This exposure includes indoor or outdoor service in a climate where concrete will not be exposed to freezing or to deicing agents.

Moderate exposure—Service in a climate where freezing is expected but where the concrete will not be continually exposed to moisture or free water for long periods prior to freezing and will not be exposed to deicing agents or other aggressive chemicals. Examples include: exterior beams, columns, walls, girders, or slabs which are not in contact with wet soil and are so located that they will not receive direct applications of deicing salts.

Severe exposure—Concrete which is exposed to deicing chemicals or other aggressive agents or where the concrete may become highly saturated by continual contact with moisture or free water prior to freezing. Examples include: pavements, bridge decks, curbs, gutters, sidewalks, canal linings, or exterior water tanks or sumps.

The use of normal amounts of air entrainment in concrete with a specified strength near or about 5000 psi may not be possible due to the fact that each added percent of air lowers the maximum strength obtainable with a given combination of materials.[18] In these cases the exposure to water, deicing salts, and freezing temperatures should be carefully evaluated. If a member is not continually wet and will not be exposed to deicing salts, lower air content values such as those given in Table 5.3.3 for moderate exposure are appropriate even though the concrete is exposed to freezing and thawing temperatures. However, for an exposure condition where the member may be saturated prior to freezing, the use of air entrainment should not be sacrificed for strength.

When trial batches are used to establish strength relationships or verify strength-producing capability of a mixture, the least favorable combination of mixing water and air content should be used. This is, the air content should be the maximum permitted or likely to occur, and the concrete should be gaged to the highest permissable slump. This will avoid developing an over-optimistic estimate of strength on the assumption that average rather than extreme conditions will prevail in the field. For information on air content recommendations, see ACI 201, 301, and 302.

5.3.4 *Step 4. Selection of water-cement ratio.* The required water-cement ratio is determined not only by strength requirements but also by factors such as durability and finishing properties. Since different aggregates and cements generally produce different strengths at the same water-cement ratio, it is highly desirable to have or develop the relationship between strength and water-cement ratio for the materials actually to be used. In the absence of such data, approximate and relatively conservative values for concrete containing Type I portland cement can be taken from Table 5.3.4(a). With typical materials, the tabulated water-cement ratios should produce the strengths shown, based on 28-day tests of specimens cured under standard laboratory conditions. The average strength selected must, of course, exceed the specified strength by a sufficient margin to keep the number of low tests within specified limits.*

For severe conditions of exposure, the water-cement ratio should be kept low even though strength requirements may be met with a higher value. Table 5.3.4(b) gives limiting values.

5.3.5 *Step 5. Calculation of cement content.* The amount of cement per unit volume of concrete is fixed by the determinations made in Steps 3 and 4 above. The required cement is equal to the estimated mixing water content (Step 3) divided by the water-cement ratio (Step 4). If, however, the specification includes a separate minimum limit on cement in addition to

*See "Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-65)."

TABLE 5.3.4(a)—RELATIONSHIPS BETWEEN WATER-CEMENT RATIO AND COMPRESSIVE STRENGTH OF CONCRETE

Compressive strength at 28 days, psi*	Water-cement ratio, by weight	
	Non-air-entrained concrete	Air-entrained concrete
6000	0.41	—
5000	0.48	0.40
4000	0.57	0.48
3000	0.68	0.59
2000	0.82	0.74

*Values are estimated average strengths for concrete containing not more than the percentage of air shown in Table 5.3.3. For a constant water-cement ratio, the strength of concrete is reduced as the air content is increased.

Strength is based on 6 x 12 in. cylinders moist-cured 28 days at 73.4 ± 3 F (23 ± 1.7 C) in accordance with Section 9(b) of ASTM C 31 for Making and Curing Concrete Compression and Flexure Test Specimens in the Field.

Relationship assumes maximum size of aggregate about ¾ to 1 in.; for a given source, strength produced for a given water-cement ratio will increase as maximum size of aggregate decreases; see Sections 3.4 and 5.3.2.

TABLE 5.3.4(b)—MAXIMUM PERMISSIBLE WATER-CEMENT RATIOS FOR CONCRETE IN SEVERE EXPOSURES*

Type of structure	Structure wet continuously or frequently and exposed to freezing and thawing†	Structure exposed to sea water or sulfates
Thin sections (railings, curbs, sills, ledges, ornamental work) and sections with less than 1 in. cover over steel	0.45	0.40‡
All other structures	0.50	0.45‡

*Based on report of ACI Committee 201, "Durability of Concrete in Service," previously cited.

†Concrete should also be air-entrained.

‡If sulfate resisting cement (Type II or Type V of ASTM C 150) is used, permissible water-cement ratio may be increased by 0.05.

requirements for strength and durability, the mixture must be based on whichever criterion leads to the larger amount of cement.

The use of pozzolanic or chemical admixtures will affect properties of both the fresh and hardened concrete.*

5.3.6 *Step 6. Estimation of coarse aggregate content.* Aggregates of essentially the same maximum size and grading will produce concrete of satisfactory workability when a given volume of coarse aggregate, on a dry-rodded basis, is used per unit volume of concrete. Appropriate values for this aggregate volume are given in Table 5.3.6. It can be seen that, for equal workability, the volume of coarse aggregate in a unit volume of concrete is dependent only on its maximum size and the fineness modulus of the fine aggregate. Differences in the amount of mortar required for workability with different aggregates, due to differences in particle shape and grading, are compensated for automatically by differences in dry-rodded void content.

The volume of aggregate, in cubic feet, on a dry-rodded basis, for a cubic yard of concrete is equal to the value from Table 5.3.6 multiplied by 27. This volume is converted to dry weight of coarse aggregate required in a cubic yard of concrete by multiplying it by the dry-rodded weight per cubic foot of the coarse aggregate.

TABLE 5.3.6—VOLUME OF COARSE AGGREGATE PER UNIT OF VOLUME OF CONCRETE

Maximum size of aggregate, in.	Volume of dry-rodded coarse aggregate* per unit volume of concrete for different fineness moduli of sand			
	2.40	2.60	2.80	3.00
⅜	0.50	0.48	0.46	0.44
½	0.59	0.57	0.55	0.53
¾	0.66	0.64	0.62	0.60
1	0.71	0.69	0.67	0.65
1½	0.75	0.73	0.71	0.69
2	0.78	0.76	0.74	0.72
3	0.82	0.80	0.78	0.76
6	0.87	0.85	0.83	0.81

*Volumes are based on aggregates in dry-rodded condition as described in ASTM C 29 for Unit Weight of Aggregate.

These volumes are selected from empirical relationships to produce concrete with a degree of workability suitable for usual reinforced construction. For less workable concrete such as required for concrete pavement construction they may be increased about 10 percent. For more workable concrete see Section 5.3.6.1.

5.3.6.1. For more workable concrete, which is sometimes required when placement is by pump or when concrete must be worked around congested reinforcing steel, it may be desirable to reduce the estimated coarse aggregate content determined using Table 5.3.6 by up to 10 percent. However, caution must be exercised to assure that the resulting slump, water-cement ratio, and strength properties of the concrete are consistent with the recommendations in Sections 5.3.1 and 5.3.4 and meet applicable project specification requirements.

5.3.7 *Step 7. Estimation of fine aggregate content.* At completion of Step 6, all ingredients of the concrete have been estimated except the fine aggregate. Its quantity is determined by difference. Either of two procedures may be employed: the "weight" method (Section 5.3.7.1) or the "absolute volume" method (Section 5.3.7.2).

5.3.7.1 If the weight of the concrete per unit volume is assumed or can be estimated from experience, the required weight of fine aggregate is simply the difference between the weight of fresh concrete and the total weight of the other ingredients. Often the unit weight of concrete is known with reasonable accuracy from previous experience with the materials. In the absence of such information, Table 5.3.7.1 can be used to make a first estimate. Even if the estimate of concrete weight per cubic yard is rough, mixture proportions will be sufficiently accurate to permit easy adjustment on the basis of trial batches as will be shown in the examples.

If a theoretically exact calculation of fresh concrete weight per cubic yard is desired, the following formula can be used:

$$U = 16.85\,G_a(100 - A) + C(1 - G_a/G_c) - W(G_a - 1) \qquad (5\text{-}1)$$

*See report of ACI Committee 212 "Admixtures for Concrete," ACI Journal, *Proceedings* V. 60, No. 11, Nov. 1963, pp. 1481-1524.

where

U = weight of fresh concrete per cubic yard, lb
G_a = weighted average specific gravity of combined fine and coarse aggregate, bulk SSD*
G_c = specific gravity of cement (generally 3.15)
A = air content, percent
W = mixing water requirement, lb per cu yd
C = cement requirement, lb per cu yd

TABLE 5.3.7.1—FIRST ESTIMATE OF WEIGHT OF FRESH CONCRETE

Maximum size of aggregate, in.	First estimate of concrete weight, lb per cu yd*	
	Non-air-entrained concrete	Air-entrained concrete
3/8	3840	3690
1/2	3890	3760
3/4	3960	3840
1	4010	3900
1 1/2	4070	3960
2	4120	4000
3	4160	4040
6	4230	4120

*Values calculated by Eq. (5-1) for concrete of medium richness (550 lb of cement per cu yd) and medium slump with aggregate specific gravity of 2.7. Water requirements based on values for 3 to 4 in. slump in Tables 5.3.3. If desired, the estimated weight may be refined as follows if necessary information is available: for each 10 lb difference in mixing water from the Table 5.3.3. values for 3 to 4 in. slump, correct the weight per cu yd 15 lb in the opposite direction; for each 100 lb difference in cement content from 550 lb, correct the weight per cu yd 15 lb in the same direction; for each 0.1 by which aggregate specific gravity deviates from 2.7, correct the concrete weight 100 lb in the same direction.

5.3.7.2 A more exact procedure for calculating the required amount of fine aggregate involves the use of volumes displaced by the ingredients. In this case, the total volume displaced by the known ingredients—water, air, cement, and coarse aggregate—is subtracted from the unit volume of concrete to obtain the required volume of fine aggregate. The volume occupied in concrete by any ingredient is equal to its weight divided by the density of that material (the latter being the product of the unit weight of water and the specific gravity of the material).

5.3.8 *Step 8. Adjustments for aggregate moisture.* The aggregate quantities actually to be weighed out for the concrete must allow for moisture in the aggregates. Generally, the aggregates will be moist and their dry weights should be increased by the percentage of water they contain, both absorbed and surface. The mixing water added to the batch must be reduced by an amount equal to the free moisture contributed by the aggregate—i.e., total moisture minus absorption.

5.3.9 *Step 9. Trial batch adjustments.* The calculated mixture proportions should be checked by means of trial batches prepared and tested in accordance with ASTM C 192, "Making and Curing Concrete Compression and Flexure Test Specimens in the Laboratory," or full-sized field batches. Only sufficient water should be used to produce the required slump regardless of the amount assumed in selecting the trial proportions. The concrete should be checked for unit weight and yield (ASTM C 138) and for air content (ASTM C 138, C 173, or C 231). It should also be carefully observed for proper workability, freedom from segregation, and finishing properties. Appropriate adjustments should be made in the proportions for subsequent batches in accordance with the following procedure.

5.3.9.1 Re-estimate the required mixing water per cubic yard of concrete by multiplying the net mixing water content of the trial batch by 27 and dividing the product by the yield of the trial batch in cubic feet. If the slump of the trial batch was not correct, increase or decrease the re-estimated amount of water by 10 lb for each required increase or decrease of 1 in. in slump.

5.3.9.2 If the desired air content (for air-entrained concrete) was not achieved, re-estimate the admixture content required for proper air content and reduce or increase the mixing water content of paragraph 5.3.9.1 by 5 lb for each 1 percent by which the air content is to be increased or decreased from that of the previous trial batch.

5.3.9.3 If estimated weight per cubic yard of fresh concrete is the basis for proportioning, re-estimate that weight by multiplying the unit weight in pounds per cubic foot of the trial batch by 27 and reducing or increasing the result by the anticipated percentage increase or decrease in air content of the adjusted batch from the first trial batch.

5.3.9.4 Calculate new batch weights starting with Step 4 (Paragraph 5.3.4), modifying the volume of coarse aggregate from Table 5.3.6 if necessary to provide proper workability.

6. SAMPLE COMPUTATIONS

6.1—Two example problems will be used to illustrate application of the proportioning procedures. The following conditions are assumed:

6.1.1 Type I non-air-entraining cement will be used and its specific gravity is assumed to be 3.1.5.†

6.1.2 Coarse and fine aggregates in each case are of satisfactory quality and are graded within limits of generally accepted specifications.‡

6.1.3 The coarse aggregate has a bulk specific gravity of 2.68† and an absorption of 0.5 percent.

*SSD indicates saturated-surface-dry basis used in considering aggregate displacement. The aggregate specific gravity used in calculations must be consistent with the moisture condition assumed in the basic aggregate batch weights—i.e., bulk dry if aggregate weights are stated on a dry basis, and bulk SSD if weights are stated on a saturated-surface-dry basis.

†The specific gravity values are not used if proportions are selected to provide a weight of concrete assumed to occupy 1 cu yd.

‡Such as the "Specifications for Concrete Aggregates," (ASTM C 33).

6.1.4 The fine aggregate has a bulk specific gravity of 2.64,* an absorption of 0.7 percent, and fineness modulus of 2.8.

6.2—*Example 1.* Concrete is required for a portion of a structure which will be below ground level in a location where it will not be exposed to severe weathering or sulfate attack. Structural considerations require it to have an average 28-day compressive strength of 3500 psi.† On the basis of information in Table 5.3.1, as well as previous experience, it is determined that under the conditions of placement to be employed, a slump of 3 to 4 in. should be used and that the available No. 4 to 1½-in. coarse aggregate will be suitable. The dry-rodded weight of coarse aggregate is found to be 100 lb per cu ft. Employing the sequence outlined in Section 5, the quantities of ingredients per cubic yard of concrete are calculated as follows:

6.2.1 *Step 1.* As indicated above, the desired slump is 3 to 4 in.

6.2.2 *Step 2.* The locally available aggregate, graded from No. 4 to 1½ in., has been indicated as suitable.

6.2.3 *Step 3.* Since the structure will not be exposed to severe weathering, non-air-entrained concrete will be used. The approximate amount of mixing water to produce a 3- to 4-in. slump in non-air-entrained concrete with 1½-in. aggregate is found from Table 5.3.3 to be 300 lb per cu yd. Estimated entrapped air is shown as 1 percent.

6.2.4 *Step 4.* From Table 5.3.4(a), the water-cement ratio needed to produce a strength of 3500 psi in non-air-entrained concrete is found to be about 0.62.

6.2.5 *Step 5.* From the information derived in Steps 3 and 4, the required cement content is found to be 300/0.62 = 484 lb per cu yd.

6.2.6 *Step 6.* The quantity of coarse aggregate is estimated from Table 5.3.6. For a fine aggregate having a fineness modulus of 2.8 and a 1½ in. maximum size of coarse aggregate, the table indicates that 0.71 cu ft of coarse aggregate, on a dry-rodded basis, may be used in each cubic foot of concrete. For a cubic yard, therefore, the coarse aggregate will be 27 × 0.71 = 19.17 cu ft. Since it weighs 100 lb per cu ft, the dry weight of coarse aggregate is 1917 lb.

6.2.7 *Step 7.* With the quantities of water, cement, and coarse aggregate established, the remaining material comprising the cubic yard of concrete must consist of sand and whatever air will be entrapped. The required sand may be determined on the basis of either weight or absolute volume as shown below:

6.2.7.1 *Weight basis.* From Table 5.3.7.1, the weight of a cubic yard of non-air-entrained concrete made with aggregate having a maximum size of 1½ in. is estimated to be 4070 lb. (For a first trial batch, exact adjustments of this value for usual differences in slump, cement factor, and aggregate specific gravity are not critical.) Weights already known are:

Water (net mixing)	300 lb
Cement	484 lb
Coarse aggregate	1917 lb (dry)‡
Total	2701 lb

The weight of sand, therefore, is estimated to be

4070 − 2701 = 1369 lb (dry)‡

6.2.7.2 *Absolute volume basis.* With the quantities of cement, water, and coarse aggregate established, and the approximate entrapped air content (as opposed to purposely entrained air) taken from Table 5.3.3, the sand content can be calculated as follows:

Volume of water	$= \dfrac{300}{62.4}$	= 4.81 cu ft
Solid volume of cement	$= \dfrac{484}{3.15 \times 62.4}$	= 2.46 cu ft
Solid volume of coarse aggregate	$= \dfrac{1917}{2.68 \times 62.4}$	= 11.46 cu ft
Volume of entrapped air	$= 0.01 \times 27$	= 0.27 cu ft
Total solid volume of ingredients except sand		= 19.00 cu ft
Solid volume of sand required	$= 27 - 19.00$	= 8.00 cu ft
Required weight of dry sand	$= 8.00 \times 2.64 \times 62.4$	= 1318 lb

6.2.7.3 Batch weights per cubic yard of concrete calculated on the two bases are compared below:

	Based on estimated concrete weight, lb	Based on absolute volume of ingredients, lb
Water (net mixing)	300	300
Cement	484	484
Coarse aggregate (dry)	1917	1917
Sand (dry)	1369	1318

6.2.8 *Step 8.* Tests indicate total moisture of 2 percent in the coarse aggregate and 6 percent in the fine aggregate. If the trial batch proportions based on assumed concrete weight are used, the adjusted aggregate weights become

*The specific gravity values are not used if proportions are selected to provide a weight of concrete assumed to occupy 1 cu yd.

†This is not the specified strength used for structural design but a higher figure expected to be produced on the average. For the method of determining the amount by which average strength should exceed design strength, see "Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-65)."

‡Aggregate absorption is disregarded since its magnitude is inconsequential in relation to other approximations.

Coarse aggregate (wet) $= 1917\ (1.02) = 1955$ lb
Fine aggregate (wet) $= 1369\ (1.06) = 1451$ lb

Absorbed water does not become part of the mixing water and must be excluded from the adjustment in added water. Thus, surface water contributed by the coarse aggregate amounts to $2 - 0.5 = 1.5$ percent; by the fine aggregate $6 - 0.7 = 5.3$ percent. The estimated requirement for added water, therefore, becomes

$$300 - 1917\,(0.015) - 1369\,(0.053) = 199 \text{ lb}$$

The estimated batch weights for a cubic yard of concrete are:

Water (to be added)	199 lb
Cement	484 lb
Coarse aggregate (wet)	1955 lb
Fine aggregate (wet)	1451 lb

6.2.9 *Step 9*. For the laboratory trial batch, it is found convenient to scale the weights down to produce 0.03 cu yd or 0.81 cu ft of concrete. Although the calculated quantity of water to be added was 5.97 lb, the amount actually used in an effort to obtain the desired 3 to 4 in. slump is 7.00 lb. The batch as mixed therefore, consists of

Water (added)	7.00 lb
Cement	14.52 lb
Coarse aggregate (wet)	58.65 lb
Fine aggregate (wet)	43.53 lb
Total	123.70 lb

The concrete has a measured slump of 2 in. and unit weight of 149.0 lb per cu ft. It is judged to be satisfactory from the standpoint of workability and finishing properties. To provide proper yield and other characteristics for future batches, the following adjustments are made:

6.2.9.1 Since the yield of the trial batch was

$$123.70/149.0 = 0.830 \text{ cu ft}$$

and the mixing water content was 7.00 (added) $+ 0.86$ on coarse aggregate $+ 2.18$ on fine aggregate $= 10.04$ lb, the mixing water required for a cubic yard of concrete with the same slump as the trial batch should be

$$\frac{10.04 \times 27}{0.830} = 327 \text{ lb}$$

As indicated in Paragraph 5.3.9.1, this amount must be increased another 15 lb to raise the slump from the measured 2 in. to the desired 3 to 4 in. range, bringing the net mixing water to 342 lb.

6.2.9.2 With the increased mixing water, additional cement will be required to provide the desired water-cement ratio of 0.62. The new cement content becomes

$$342/0.62 = 552 \text{ lb}$$

6.2.9.3 Since workability was found to be satisfactory, the quantity of coarse aggregate per unit volume of concrete will be maintained the same as in the trial batch. The amount of coarse aggregate per cubic yard becomes

$$\frac{58.65}{0.83} \times 27 = 1908 \text{ lb wet}$$

which is

$$\frac{1908}{1.02} = 1871 \text{ lb dry}$$

and

$$1871\ (1.005) = 1880 \text{ SSD*}$$

6.2.9.4 The new estimate for the weight of a cubic yard of concrete is $149.0 \times 27 = 4023$ lb. The amount of sand required is, therefore,

$$4023 - (342 + 552 + 1880) = 1249 \text{ lb SSD}$$

or

$$1249/1.007 = 1240 \text{ lb dry}$$

The adjusted basic batch weights per cubic yard of concrete are

Water (net mixing)	342 lb
Cement	552 lb
Coarse aggregate (dry)	1871 lb
Fine aggregate (dry)	1240 lb

6.2.10 Adjustments of proportions determined on an absolute volume basis follow a procedure similar to that just outlined. The steps will be given without detailed explanation:

6.2.10.1 Quantities used in nominal 0.81 cu ft batch are

Water (added)	7.00 lb
Cement	14.52 lb
Coarse aggregate (wet)	58.65 lb
Fine aggregate (wet)	41.91 lb
Total	122.08 lb

Measured slump 2 in.; unit weight 149.0 lb per cu ft; yield $122.08/149.0 = 0.819$ cu ft; workability o.k.

6.2.10.2 Re-estimated water for same slump as trial batch:

$$\frac{27\,(7.00 + 0.86 + 2.09)}{0.819} = 328 \text{ lb}$$

Mixing water required for slump of 3 to 4 in.:

$$328 + 15 = 343 \text{ lb}$$

6.2.10.3 Adjusted cement content for increased water:

$$343/0.62 = 553 \text{ lb}$$

6.2.10.4 Adjusted coarse aggregate requirement:

$$\frac{58.65}{0.819} \times 27 = 1934 \text{ lb wet}$$

or

$$1934/1.02 = 1896 \text{ lb dry}$$

*Saturated-surface-dry

6.2.10.5 The volume of ingredients other than air in the original trial batch was

Water	$\frac{9.95}{62.4}$	$= 0.159$ cu ft
Cement	$\frac{14.52}{3.15 \times 62.4}$	$= 0.074$ cu ft
Coarse aggregate	$\frac{57.50}{2.68 \times 62.4}$	$= 0.344$ cu ft
Fine aggregate	$\frac{39.54}{2.64 \times 62.4}$	$= 0.240$ cu ft
Total		$= 0.817$ cu ft

Since the yield was 0.819 cu ft, the air content was

$$\frac{0.819 - 0.817}{0.819} = 0.2 \text{ percent}$$

With the proportions of all components except fine aggregate established, the determination of adjusted cubic yard batch quantities can be completed as follows:

Volume of water	$= \frac{343}{62.4}$	$=$	5.50 cu ft
Volume of cement	$= \frac{553}{3.15 \times 62.4}$	$=$	2.81 cu ft
Volume of air	$= 0.002 \times 27$	$=$	0.05 cu ft
Volume of coarse aggregate	$= \frac{1896}{2.68 \times 62.4}$	$=$	11.34 cu ft
Total volume exclusive of fine aggregate		$=$	19.70 cu ft
Volume of fine aggregate required	$= 27 - 19.70$	$=$	7.30 cu ft
Weight of fine aggregate (dry basis)	$= 7.30 \times 2.64 \times 62.4$	$=$	1203 lb

The adjusted basic batch weights per cubic yard of concrete, then, are:

Water (net mixing)	343 lb
Cement	553 lb
Coarse aggregate (dry)	1896 lb
Fine aggregate (dry)	1203 lb

These differ only slightly from those given in Paragraph 6.2.9.4 for the method of assumed concrete weight. Further trials or experience might indicate small additional adjustments for either method.

6.3—*Example 2.* Concrete is required for a heavy bridge pier which will be exposed to fresh water in a severe climate. An average 28-day compressive strength of 3000 psi will be required. Placement conditions permit a slump of 1 to 2 in. and the use of large aggregate, but the only economically available coarse aggregate of satisfactory quality is graded from No. 4 to 1 in. and this will be used. Its dry-rodded weight is found to be 95 lb per cu ft. Other characteristics are as indicated in Section 6.1.

The calculations will be shown in skeleton form only. Note that confusion is avoided if all steps of Section 5 are followed even when they appear repetitive of specified requirements.

6.3.1 *Step 1.* The desired slump is 1 to 2 in.

6.3.2 *Step 2.* The locally available aggregate, graded from No. 4 to 1 in., will be used.

6.3.3 *Step 3.* Since the structure will be exposed to severe weathering, air-entrained concrete will be used. The approximate amount of mixing water to produce a 1 to 2-in. slump in air-entrained concrete with 1-in. aggregate is found from Table 5.3.3 to be 270 lb per cu yd. The recommended air content is 5 percent.

6.3.4 *Step 4.* From Table 5.3.4(a), the water-cement ratio needed to produce a strength of 3000 psi in air-entrained concrete is estimated to be about 0.59. However, reference to Table 5.3.4(b) reveals that, for the severe weathering exposure anticipated, the water-cement ratio should not exceed 0.50. This lower figure must govern and will be used in the calculations.

6.3.5 *Step 5.* From the information derived in Steps 3 and 4, the required cement content is found to be $270/0.50 = 540$ lb per cu yd.

6.3.6. *Step 6.* The quantity of coarse aggregate is estimated from Table 5.3.6. With a fine aggregate having a fineness modulus of 2.8 and a 1 in. maximum size of coarse aggregate, the table indicates that 0.67 cu ft of coarse aggregate, on a dry-rodded basis, may be used in each cubic foot of concrete. For a cubic yard, therefore, the coarse aggregate will be $27 \times 0.67 = 18.09$ cu ft. Since it weighs 95 lb per cu ft, the dry weight of coarse aggregate is $18.09 \times 95 = 1719$ lb.

6.3.7 *Step 7.* With the quantities of water, cement and coarse aggregate established, the remaining material comprising the cubic yard of concrete must consist of sand and air. The required sand may be determined on the basis of either weight or absolute volume as shown below.

6.3.7.1 *Weight basis.* From Table 5.3.7.1, the weight of a cubic yard of air-entrained concrete made with aggregate of 1 in. maximum size is estimated to be 3900 lb. (For a first trial batch, exact adjustments of this value for differences in slump, cement factor, and aggregate specific gravity are not critical.) Weights already known are:

Water (net mixing)	270 lb
Cement	540 lb
Coarse aggregate (dry)	1719 lb
Total	2529 lb

The weight of sand, therefore, is estimated to be

$$3900 - 2529 = 1371 \text{ lb (dry)}$$

6.3.7.2 *Absolute volume basis.* With the quantities of cement, water, air, and coarse aggregate established, the sand content can be calculated as follows:

Volume of water	$= \dfrac{270}{62.4}$	= 4.33 cu ft
Solid volume of cement	$= \dfrac{540}{3.15 \times 62.4}$	= 2.75 cu ft
Solid volume of coarse aggregate	$= \dfrac{1719}{2.68 \times 62.4}$	= 10.28 cu ft
Volume of air	$= 0.05 \times 27$	= 1.35 cu ft
Total volume of ingredients except sand		= 18.71 cu ft
Solid volume of sand required	$= 27 - 18.71$	= 8.29 cu ft
Required weight of dry sand	$= 8.29 \times 2.64 \times 62.4$	= 1366 lb

6.3.7.3 Batch weights per cubic yard of concrete calculated on the two bases are compared below:

	Based on estimated concrete weight, lb	Based on absolute volume of ingredients, lb
Water (net mixing)	270	270
Cement	540	540
Coarse aggregate (dry)	1719	1719
Sand (dry)	1371	1366

6.3.8 *Step 8.* Tests indicate total moisture of 3 percent in the coarse aggregate and 5 percent in the fine aggregate. If the trial batch proportions based on assumed concrete weight are used, the adjusted aggregate weights become

Coarse aggregate (wet) $= 1719\,(1.03) = 1771$ lb
Fine aggregate (wet) $= 1371\,(1.05) = 1440$ lb

Absorbed water does not become part of the mixing water and must be excluded from the adjustment in added water. Thus, surface water contributed by the coarse aggregate amounts to $3 - 0.5 = 2.5$ percent; by the fine aggregate $5 - 0.7 = 4.3$ percent. The estimated requirement for added water, therefore, becomes

$$270 - 1719\,(0.025) - 1371\,(0.043) = 168 \text{ lb}$$

The estimated batch weights for a cubic yard of concrete are:

Water (to be added)	168 lb
Cement	540 lb
Coarse aggregate (wet)	1771 lb
Fine aggregate (wet)	1440 lb
Total	3919 lb

6.3.9 *Step 9.* For the laboratory trial batch, the weights are scaled down to produce 0.03 cu yd or 0.81 cu ft of concrete. Although the calculated quantity of water to be added was 5.04 lb the amount actually used is in an effort to obtain the desired 1 to 2-in. slump is 4.50 lb. The batch as mixed, therefore, consists of

Water (added)	4.50 lb
Cement	16.20 lb
Coarse aggregate (wet)	53.13 lb
Fine aggregate (wet)	43.20 lb
Total	117.03 lb

The concrete has a measured slump of 2 in., unit weight of 141.8 lb per cu ft, and air content of 6.5 percent. It is judged to be slightly oversanded for the easy placement condition involved. To provide proper yield and other characteristics for future batches, the following adjustments are made:

6.3.9.1 Since the yield of the trial batch was

$$117.03/141.8 = 0.825 \text{ cu ft}$$

and the mixing water content was 4.50 (added) + 1.29 on coarse aggregate + 1.77 on fine aggregate = 7.56 lb the mixing water required for a cubic yard of concrete with the same slump as the trial batch should be

$$\frac{7.56 \times 27}{0.825} = 247 \text{ lb}$$

The slump was satisfactory but, since the air content was too high by 1.5 percent, more water will be needed for proper slump when the air content is corrected. As indicated in Paragraph 5.3.9.2, the mixing water should be increased roughly 5×1.5 or about 8 lb, bringing the new estimate to 255 lb per cu yd.

6.3.9.2 With the decreased mixing water, less cement will be required to provide the desired water-cement ratio of 0.5. The new cement content becomes

$$255/0.5 = 510 \text{ lb}$$

6.3.9.3 Since the concrete was found to be oversanded, the quantity of coarse aggregate per unit volume will be increased 10 percent, to 0.74, in an effort to correct the condition. The amount of coarse aggregate per cubic yard becomes

$$0.74 \times 27 \times 95 = 1898 \text{ lb dry}$$

or

$$1898 \times 1.03 = 1955 \text{ lb wet}$$

and

$$1898 \times 1.005 = 1907 \text{ lb SSD*}$$

*Saturated-surface-dry

6.3.9.4 The new estimate for the weight of the concrete with 1.5 percent less air is 141.8/0.985 = 144.0 lb per cu ft or $144.0 \times 27 = 3888$ lb per cu yd. The weight of sand, therefore, is

$$3888 - (255 + 510 + 1907) = 1216 \text{ lb SSD*}$$

or

$$1216/1.007 = 1208 \text{ lb dry}$$

The adjusted basic batch weights per cubic yard of concrete are

Water (net mixing)	255 lb
Cement	510 lb
Coarse aggregate (dry)	1898 lb
Fine aggregate (dry)	1208 lb

Admixture dosage must be reduced to provide the desired air content.

6.3.10 Adjustments of proportions determined on an absolute volume basis would follow the procedure outlined in Paragraph 6.2.10 which will not be repeated for this example.

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*Saturated-surface-dry

APPENDICES

APPENDIX 1—METRIC SYSTEM ADAPTATION

A1.1—Procedures outlined in this recommended practice have been presented using British (United States customary) units of measurement. The principles are equally applicable in the metric system with proper adaptation of units. This Appendix provides all of the information necessary to apply the proportioning procedure using International SI (metric) measurements. Table A1.1 gives relevant conversion factors. A numerical example is presented in Appendix 2.

A1.2—For convenience of reference, numbering of subsequent paragraphs in this Appendix corresponds to the body of the report except that the designation "A1" is prefixed. All tables have been converted and reproduced. Descriptive portions are included only where use of the metric system requires a change in a procedure or formula. To the extent practicable, conversions to metric units have been made in such a way that values are realistic in terms of usual practice and significance of numbers. For example, aggregate and sieve sizes in the metric tables are ones commonly used in Europe. Thus, there is not always a precise mathematical correspondence between British and metric values in corresponding tables.

A1.5.2 *Steps in calculating proportions.* Except as discussed below, the methods for arriving at quantities of ingredients for a unit volume of concrete are essentially the same when metric units are employed as when British units are employed. The main difference is that the unit volume of concrete becomes the cubic meter and numerical values must be taken from the proper "A1" table instead of the one referred to in the text.

A1.5.2.1 *Step 1. Choice of slump.* See Table A1.5.2.1.

A1.5.2.2 *Step 2. Choice of maximum size of aggregate.*

A1.5.2.3 *Step 3. Estimation of mixing water and air content.* See Table A1.5.2.3.

A1.5.2.4 *Step 4. Selection of water-cement ratio.* See Table A1.5.2.4.

A1.5.2.5 *Step 5. Calculation of cement content.*

A1.5.2.6 *Step 6. Estimation of coarse aggregate content.* The dry weight of coarse aggregate required for a cubic meter of concrete is equal to the value from Table A1.5.2.6 multiplied by the dry-rodded unit weight of the aggregate in kilograms per cubic meter.

TABLE A1.1—CONVERSION FACTORS, BRITISH TO METRIC UNITS*

Quantity	British (U.S. customary) unit	SI† (Metric) unit	Conversion factor (Ratio: British/SI)
Length	inch (in.)	centimeter (cm)	2.540
	inch (in.)	millimeter (mm)	25.40
Volume	cubic foot (ft^3)	cubic meter (m^3)	0.02832
	cubic yard (yd^3)	cubic meter (m^3)	0.7646
Mass	pound (lb)	kilogram (kg)	0.4536
Stress	pounds per square inch (psi)	kilograms force per square centimeter (kgf/cm^2)	0.0703
Density	pounds per cubic foot (lb/ft^3)	kilograms per cubic meter (kg/m^3)	16.02
	pounds per cubic yard (lb/yd^3)	kilograms per cubic meter (kg/m^3)	0.5933
Temperature	degrees Fahrenheit (F)	degrees Centigrade (C)	‡

*Gives names (and abbreviations) of measurement units in the British (U.S. customary) system as used in the body of this report and in the S.I. (metric) system, along with multipliers for converting the former to the latter. From "ASTM Metric Practice Guide" (2nd Edition, 1966).

†Systéme International d'Unités

‡$C = (F - 32)/1.8$.

TABLE A1.5.2.1—RECOMMENDED SLUMPS FOR VARIOUS TYPES OF CONSTRUCTION (METRIC)

Types of construction	Slump, cm Maximum*	Slump, cm Minimum
Reinforced foundation walls and footings	8	2
Plain footings, caissons, and substructure walls	8	2
Beams and reinforced walls	10	2
Building columns	10	2
Pavements and slabs	8	2
Heavy mass concrete	8	2

*May be increased 2 cm for methods of consolidation other than vibration.

TABLE A1.5.2.3—APPROXIMATE MIXING WATER REQUIREMENTS FOR DIFFERENT SLUMPS AND MAXIMUM SIZES OF AGGREGATES (METRIC)*

Slump, cm.	Water, kg/m³ of concrete for indicated maximum sizes of aggregate in mm							
	10	12.5	20	25	40	50†	70†	150†
Non-air-entrained concrete								
3 to 5	205	200	185	180	160	155	145	125
8 to 10	225	215	200	195	175	170	160	140
15 to 18	240	230	210	205	185	180	170	—
Approximate amount of entrapped air in non-air-entrained concrete, percent	3	2.5	2	1.5	1	0.5	0.3	0.2
Air-entrained concrete								
3 to 5	180	175	165	160	145	140	135	120
8 to 10	200	190	180	175	160	155	150	135
15 to 18	215	205	190	185	170	165	160	—
Recommended average total air content, percent	8	7	6	5	4.5	4	3.5	3

*These quantities of mixing water are for use in computing cement factors for trial batches. They are maxima for reasonably well-shaped angular coarse aggregates graded within limits of accepted specifications.

†The slump values for concrete containing aggregate larger than 40 mm are based on slump tests after removal of particles larger than 40 mm by wet-screening.

TABLE A1.5.2.4(a)—RELATIONSHIPS BETWEEN WATER-CEMENT RATIO AND COMPRESSIVE STRENGTH OF CONCRETE (METRIC)

Compressive strength at 28 days, kgf/cm²*	Water-cement ratio, by weight: Non-air-entrained concrete	Air-entrained concrete
450	0.38	—
400	0.43	—
350	0.48	0.40
300	0.55	0.46
250	0.62	0.53
200	0.70	0.61
150	0.80	0.71

*Values are estimated average strengths for concrete containing not more than the percentage of air shown in Table A1.5.2.3. For a constant water-cement ratio, the strength of concrete is reduced as the air content is increased.

Strength is based on 15 x 30 cm cylinders moist-cured 28 days at 23 ± 1.7 C in accordance with Section 9(b) of ASTM C 31 for Making and Curing Concrete Compression and Flexure Test Specimens in the Field. Cube strengths will be higher by approximately 20 percent.

Relationship assumes maximum size of aggregate about 20 to 30 mm; for a given source, strength produced by a given water-cement ratio will increase as maximum size decreases; see Sections 3.4 and 5.3.2.

TABLE A1.5.2.4(b)—MAXIMUM PERMISSIBLE WATER-CEMENT RATIOS FOR CONCRETE IN SEVERE EXPOSURES (METRIC)*

Type of Structure	Structure wet continuously or frequently and exposed to freezing and thawing†	Structure exposed to sea water or sulfates
Thin sections (railings, curbs, sills, ledges, ornamental work) and sections with less than 3 cm cover over steel	0.45	0.40‡
All other structures	0.50	0.45‡

*Based on the report of ACI Committee 201, "Durability of Concrete in Service," previously cited.

†Concrete should also be air-entrained.

‡If sulfate resisting cement (Type II or Type V of ASTM C 150) is used, permissible water-cement ratio may be increased by 0.05.

A1.5.2.7 *Step 7. Estimation of fine aggregate content.* In the metric system, the formula for calculation of fresh concrete weight per cubic meter is:

$$U_M = 10G_a(100 - A) + C_M(1 - G_a/G_c) - W_M(G_a - 1)$$

where

U_M = weight of fresh concrete, kg/m³
G_a = weighted average specific gravity of combined fine and coarse aggregate, bulk, SSD
G_c = specific gravity of cement (generally 3.15)
A = air content, percent
W_M = mixing water requirement, kg/m³
C_M = cement requirement, kg/m³

A1.5.2.9 *Trial batch adjustments.* The following "rules of thumb" may be used to arrive at closer approximations of unit batch quantities based on results for a trial batch:

A1.5.2.9.1 The estimated mixing water to produce the same slump as the trial batch will be equal to the net amount of mixing water used divided by the yield of the trial batch in m³. If slump of the trial batch was not correct, increase or decrease the re-estimated water content by 2 kg/m³ of concrete for each increase or decrease of 1 cm in slump desired.

A1.5.2.9.2 To adjust for the effect of incorrect air content in a trial batch of air-entrained concrete on slump, reduce or increase the mixing water

TABLE A1.5.2.6—VOLUME OF COARSE AGGREGATE PER UNIT OF VOLUME OF CONCRETE (METRIC)

Maximum size of aggregate, mm	Volume of dry-rodded coarse aggregate* per unit volume of concrete for different fineness moduli† of sand			
	2.40	2.60	2.80	3.00
10	0.50	0.48	0.46	0.44
12.5	0.59	0.57	0.55	0.53
20	0.66	0.64	0.62	0.60
25	0.71	0.69	0.67	0.65
40	0.76	0.74	0.72	0.70
50	0.78	0.76	0.74	0.72
70	0.81	0.79	0.77	0.75
150	0.87	0.85	0.83	0.81

*Volumes are based on aggregates in dry-rodded condition as described in ASTM C 29 for Unit Weight of Aggregate.

These volumes are selected from empirical relationships to produce concrete with a degree of workability suitable for usual reinforced construction. For less workable concrete such as required for concrete pavement construction they may be increased about 10 percent. For more workable concrete, such as may sometimes be required when placement is to be by pumping, they may be reduced up to 10 percent.

†Fineness modulus of sand = sum of ratios (cumulative) retained on sieves with square openings of 0.149, 0.297, 0.595, 1.19, 2.38, and 4.76 mm.

TABLE A1.5.2.7.1—FIRST ESTIMATE OF WEIGHT OF FRESH CONCRETE (METRIC)

Maximum size of aggregate, mm	First estimate of concrete weight, kg/m³*: Non-air-entrained concrete	Air-entrained concrete
10	2285	2190
12.5	2315	2235
20	2355	2280
25	2375	2315
40	2420	2355
50	2445	2375
70	2465	2400
150	2505	2435

*Values calculated by Eq. (A1.5.2.7) for concrete of medium richness (330 kg of cement per m³) and medium slump with aggregate specific gravity of 2.7. Water requirements based on values for 8 to 10 cm slump in Table A1.5.2.3. If desired, the estimate of weight may be refined as follows if necessary information is available: for each 5 kg difference in mixing water from the Table A1.5.2.3 values for 8 to 10 cm slump, correct the weight per m³ 8 kg in the opposite direction; for each 20 kg difference in cement content from 330 kg, correct the weight per m³ 3 kg in the same direction; for each 0.1 by which aggregate specific gravity deviates from 2.7, correct the concrete weight 70 kg in the same direction.

content of A1.5.2.9.1 by 3 kg/m^3 of concrete for each 1 percent by which the air content is to be increased or decreased from that of the trial batch.

A1.5.2.9.3 The re-estimated unit weight of the fresh concrete for adjustment of trial batch proportions is equal to the unit weight in kg/m^3 measured on the trial batch, reduced or increased by the percentage increase or decrease in air content of the adjusted batch from the first trial batch.

APPENDIX 2 —

EXAMPLE PROBLEM IN METRIC SYSTEM

A2.1—Example 1. Example 1 presented in Section 6.2 will be solved here using metric units of measure. Required average strength will be 250 kgf/cm^2 with slump of 8 to 10 cm. The coarse aggregate has a maximum size of 40 mm and dry-rodded weight of 1600 kg/m^3. As stated in Section 6.1, other properties of the ingredients are: cement—Type I with specific gravity of 3.15; coarse aggregate—bulk specific gravity 2.68 and absorption 0.5 percent; fine aggregate—bulk specific gravity 2.64, absorption 0.7 percent, and fineness modulus 2.8.

A2.2— All steps of Section 5.3 should be followed in sequence to avoid confusion, even though they sometimes merely restate information already given.

A2.2.1 *Step 1.* The slump is required to be 8 to 10 cm.

A2.2.2 *Step 2.* The aggregate to be used has a maximum size of 40 mm.

A2.2.3 *Step 3.* The concrete will be non-air-entrained since the structure is not to be exposed to severe weathering. From Table A1.5.2.3, the estimated mixing water for a slump of 8 to 10 cm in non-air-entrained concrete made with 40-mm aggregate is found to be 175 kg/m^3.

A2.2.4 *Step 4.* The water-cement ratio for non-air-entrained concrete with a strength of 250 kgf/cm^2 is found from Table A1.5.2.4(a) to be 0.62.

A2.2.5 *Step 5.* From the information developed in Steps 3 and 4, the required cement content is found to be $175/0.62 = 282$ kg/m^3.

A2.2.6 *Step 6.* The quantity of coarse aggregate is estimated from Table A1.5.2.6. For a fine aggregate having a fineness modulus of 2.8 and a 40 mm maximum size of coarse aggregate, the table indicates that 0.72 m^3 of coarse aggregate, on a dry-rodded basis, may be used in each cubic meter of concrete. The required dry weight is, therefore, $0.72 \times 1600 = 1152$ kg.

A2.2.7 *Step 7.* With the quantities of water, cement and coarse aggregate established, the remaining material comprising the cubic meter of concrete must consist of sand and whatever air will be entrapped. The required sand may be determined on the basis of either weight or absolute volume as shown below:

A2.2.7.1 *Weight Basis.* From Table A1.5.2.7.1, the weight of a cubic meter of non-air-entrained concrete made with aggregate having a maximum size of 40 mm is estimated to be 2420 kg. (For a first trial batch, exact adjustments of this value for usual differences in slump, cement factor, and aggregate specific gravity are not critical.) Weights already known are:

Water (net mixing)	175 kg
Cement	282 kg
Coarse aggregate	1152 kg
Total	1609 kg

The weight of sand, therefore, is estimated to be

$$2420 - 1609 = 811 \text{ kg}$$

A2.2.7.2 *Absolute volume basis.* With the quantities of cement, water, and coarse aggregate established, and the approximate entrapped air content (as opposed to purposely entrained air) of 1 percent determined from Table A1.5.2.3, the sand content can be calculated as follows:

Volume of water	$= \dfrac{175}{1000}$	0.175 m^3
Solid volume of cement	$= \dfrac{282}{3.15 \times 1000}$	0.090 m^3
Solid volume of coarse aggregate	$= \dfrac{1152}{2.68 \times 1000}$	0.430 m^3
Volume of entrapped air	$= 0.01 \times 1.000$	0.010 m^3
Total solid volume of ingredients except sand		0.705 m^3
Solid volume of sand required	$= 1.000 - 0.705$	0.295 m^3
Required weight of dry sand	$= 0.295 \times 2.64 \times 1000$	779 kg

A2.2.7.3 Batch weights per cubic meter of concrete calculated on the two bases are compared below:

	Based on estimated concrete weight, kg	Based on absolute volume of ingredients, kg
Water (net mixing)	175	175
Cement	282	282
Coarse aggregate (dry)	1152	1152
Sand (dry)	811	779

A2.2.8 *Step 8.* Tests indicate total moisture of 2 percent in the coarse aggregate and 6 percent in the fine aggregate. If the trial batch proportions based on assumed concrete weight are used, the adjusted aggregate weights become

$$\text{Coarse aggregate (wet)} = 1152(1.02) = 1175 \text{ kg}$$
$$\text{Fine aggregate (wet)} = 811(1.06) = 860 \text{ kg}$$

Absorbed water does not become part of the mixing water and must be excluded from the adjustment in added water. Thus, surface water contributed by the coarse aggregate amounts to $2 - 0.5 = 1.5$ percent; by the fine aggregate $6 - 0.7 = 5.3$ percent. The estimated requirement for added water, therefore, becomes

$$175 - 1152(0.015) - 811(0.053) = 115 \text{ kg}$$

The estimated batch weights for a cubic meter of concrete are:

Water (to be added)	115 kg
Cement	282 kg
Coarse aggregate (wet)	1175 kg
Fine aggregate (wet)	860 kg
Total	2432 kg

A2.2.9 *Step 9.* For the laboratory trial batch, it is found convenient to scale the weights down to produce 0.02 m^3 of concrete. Although the calculated quantity of water to be added was 2.30 kg, the amount actually used in an effort to obtain the desired 8 to 10 cm slump is 2.70 kg. The batch as mixed, therefore, consists of

Water (added)	2.70 kg
Cement	5.64 kg
Coarse aggregate (wet)	23.50 kg
Fine aggregate (wet)	17.20 kg
Total	49.04 kg

The concrete has a measured slump of 5 cm and unit weight of 2390 kg/m^3. It is judged to be satisfactory from the standpoint of workability and finishing properties. To provide proper yield and other characteristics for future batches, the following adjustments are made:

A2.2.9.1 Since the yield of the trial batch was

$$49.04/2390 = 0.0205 \text{ m}^3$$

and the mixing water content was 2.70 (added) + 0.34 (on coarse aggregate) + 0.86 (on fine aggregate) = 3.90 kg, the mixing water required for a cubic meter of concrete with the same slump as the trial batch should be

$$\frac{3.90}{0.0205} = 190 \text{ kg}$$

As indicated in A1.5.2.9.1, this amount must be increased another 8 kg to raise the slump from the measured 5 cm to the desired 8 to 10 cm range, bringing the total mixing water to 198 kg.

A2.2.9.2 With the increased mixing water, additional cement will be required to provide the desired water-cement ratio of 0.62. The new cement content becomes

$$198/0.62 = 319 \text{ kg}$$

A2.2.9.3 Since workability was found to be satisfactory, the quantity of coarse aggregate per unit volume of concrete will be maintained the same as in the trial batch. The amount of coarse aggregate per cubic meter becomes

$$\frac{23.50}{0.0205} = 1146 \text{ kg wet}$$

which is

$$\frac{1146}{1.02} = 1124 \text{ kg dry}$$

and

$$1124 \times 1.005 = 1130 \text{ kg SSD*}$$

A2.2.9.4 The new estimate for the weight of a cubic meter of concrete is the measured unit weight of 2390 kg/m^3. The amount of sand required is, therefore

$$2390 - (198 + 319 + 1130) = 743 \text{ kg SSD}$$

or

$$743/1.007 = 738 \text{ kg dry}$$

The adjusted basic batch weights per cubic meter of concrete are

Water (net mixing)	198 kg
Cement	319 kg
Coarse aggregate (dry)	1124 kg
Fine aggregate (dry)	738 kg

A2.2.10 Adjustments of proportions determined on an absolute volume basis follow a procedure similar to that just outlined. The steps will be given without detailed explanation:

A2.2.10.1 Quantities used in the nominal 0.02 m^3 batch are

Water (added)	2.70 kg
Cement	5.64 kg
Coarse aggregate (wet)	23.50 kg
Fine aggregate (wet)	16.51 kg
Total	48.35 kg

Measured slump 5 cm; unit weight 2390 kg/m^3; yield 48.35/2390 = 0.0202 m^3; workability o.k.

A2.2.10.2 Re-estimated water for same slump as trial batch:

$$\frac{2.70 + 0.34 + 0.83}{0.0202} = 192 \text{ kg}$$

Mixing water required for slump of 8 to 10 cm:

$$192 + 8 = 200 \text{ kg}$$

A2.2.10.3 Adjusted cement content for increased water:

$$200/0.62 = 323 \text{ kg}$$

A2.2.10.4 Adjusted coarse aggregate requirement:

$$\frac{23.50}{0.0202} = 1163 \text{ kg wet}$$

or

$$1163/1.02 = 1140 \text{ kg dry}$$

A2.2.10.5 The volume of ingredients other than air in the original trial batch was

Water	$\frac{3.87}{1000}$	= 0.0039 m^3
Cement	$\frac{5.64}{3.15 \times 1000}$	= 0.0018 m^3
Coarse aggregate	$\frac{23.04}{2.68 \times 1000}$	= 0.0086 m^3
Fine aggregate	$\frac{15.58}{2.64 \times 1000}$	= 0.0059 m^3
Total		0.0202 m^3

Since the yield was also 0.0202 m^3, there was no air in the concrete detectable within the precision of the unit weight test and significant figures of the calculations. With the proportions of all components except fine aggregate established, the determination of adjusted cubic yard batch quantities can be completed as follows:

Volume of water	$= \frac{200}{1000}$	= 0.200 m^3
Volume of cement	$= \frac{323}{3.15 \times 1000}$	= 0.103 m^3
Allowance for volume of air		= 0.000 m^3
Volume of coarse aggregate	$= \frac{1140}{2.68 \times 1000}$	= 0.425 m^3
Total volume exclusive of fine aggregate		= 0.728 m^3
Volume of fine aggregate required	= 1.000 − 0.728	= 0.272 m^3
Weight of fine aggregate (dry basis)	= 0.272 × 2.64 × 1000	= 718 kg

The adjusted basic batch weights per cubic meter of concrete, then, are:

Water (net mixing)	200 kg
Cement	323 kg
Coarse aggregate (dry)	1140 kg
Fine aggregate (dry)	718 kg

These differ only slightly from those given in Paragraph A2.2.9.4 for the method of assumed concrete weight. Further trials or experience might indicate small additional adjustments for either method.

*Saturated-surface-dry

APPENDIX 3—LABORATORY TESTS

A3.1—Selection of concrete mix proportions can be accomplished effectively from results of laboratory tests which determine basic physical properties of materials to be used, establish relationships between water-cement ratio, air content, cement content, and strength, and which furnish information on the workability characteristics of various combinations of ingredient materials. The extent of investigation desirable for any given job will depend on its size and importance and on the service conditions involved. Details of the laboratory program will also vary, depending on facilities available and on individual preferences.

A3.2—Properties of cement

A3.2.1 Physical and chemical characteristics of cement influence the properties of hardened concrete. However, the only property of cement used directly in computation of concrete mix proportions is specific gravity. The specific gravity of portland cements of the types covered by ASTM C 150 and C 175 may usually be assumed to be 3.15 without introducing appreciable error in mix computations. For other types such as the blended hydraulic cements of ASTM C 595, the specific gravity for use in volume calculations should be determined by test.

A3.2.2 A sample of cement should be obtained from the mill which will supply the job, or preferably from the concrete supplier. The sample should be ample for tests contemplated with a liberal margin for additional tests that might later be considered desirable. Cement samples should be shipped in airtight containers, or at least in moisture-proof packages.

A3.3—Properties of aggregate

A3.3.1 Sieve analysis, specific gravity, absorption, and moisture content of both fine and coarse aggregate and dry-rodded unit weight of coarse aggregate are physical properties useful for mix computations. Other tests which may be desirable for large or special types of work include petrographic examination and tests for chemical reactivity, soundness, durability, resistance to abrasion, and various deleterious substances. Such tests yield information of value in judging the long-range serviceability of concrete.

A3.3.2 Aggregate gradation as measured by the sieve analysis is a major factor in determining unit water requirement, proportions of coarse aggregate and sand, and cement content for satisfactory workability. Numerous "ideal" aggregate grading curves have been proposed, and these, tempered by practical considerations, have formed the basis for typical sieve analysis requirements in concrete standards. ASTM C 33, "Specification for Concrete Aggregates," provides a selection of sizes and gradings suitable for most concrete. Additional workability realized by use of air-entrainment permits, to some extent, the use of less restrictive aggregate gradations.

A3.3.3 Samples for concrete mix tests should be representative of aggregate available for use in the work. For laboratory tests, the coarse aggregates should be separated into required size fractions and reconstituted at the time of mixing to assure representative grading for the small test batches. Under some conditions, for work of important magnitude, laboratory investigation may involve efforts to overcome grading deficiencies of the available aggregates. Undesirable sand grading may be corrected by: (1) separation of the sand into two or more size fractions and recombining in suitable proportions; (2) increasing or decreasing the quantity of certain sizes to balance the grading; or (3) reducing excess coarse material by grinding or crushing. Undesirable coarse-aggregate gradings may be corrected by: (1) crushing excess coarser fractions; (2) wasting sizes that occur in excess; (3) supplementing deficient sizes from other sources; or (4) a combination of these methods. Whatever grading adjustments are made in the laboratory should be practical and economically justified from the standpoint of job operation. Usually, required aggregate grading should be consistent with that of economically available materials.

A3.4—Trial batch series

A3.4.1 The tabulated relationships in the body of this report may be used to make rough estimates of batch quantities for a trial mix. However, they are too generalized to apply with a high degree of accuracy to a specific set of materials. If facilities are available, therefore, it is advisable to make a series of concrete tests to establish quantitative relationships for the materials to be used. An illustration of such a test program is shown in Table A3.4.1.

A3.4.2 First, a batch of medium cement content and usable consistency is proportioned by the described methods. In preparing Mix No. 1, an amount of water is used which will produce the desired slump even if this differs from the estimated requirement. The fresh concrete is tested for slump and unit weight and observed closely for workability and finishing characteristics. In the example, the yield is too high and the concrete is judged to contain an excess of sand.

A3.4.3 Mix No. 2 is prepared, adjusted to correct the errors in Mix No. 1, and the testing and evaluation repeated. In this case, the desired properties are achieved within close tolerances and cylinders are molded to check the compressive strength. The information derived so far can now be used to select proportions for a series of additional mixes, No. 3 to 6, with cement contents above and below that of Mix No. 2, encompassing the range likely to be needed. Reasonable refinement in these batch weights can be achieved with the help of corrections given in the notes to Table 5.3.7.1.

A3.4.4 Mix No. 2 to 6 provide the background, including the relationship of strength to water-cement

TABLE A3.4.1—TYPICAL TEST PROGRAM TO ESTABLISH CONCRETE-MAKING PROPERTIES OF LOCAL MATERIALS

	Cubic yard batch quantities, lb						Concrete characteristics				
				Water							
Mix No.	Cement	Sand	Coarse Aggregate	Estimated	Used	Total used	Slump, in.	Unit wt., lb per cu ft	Yield, cu ft	28-day Compressive strength, psi	Workability
1	500	1375	1810	325	350	4035	4	147.0	27.45	—	Oversanded
2	500	1250	1875	345	340	3965	3	147.0	26.97	3350	o.k.
3	400	1335	1875	345	345	3955	4.5	145.5	27.18	2130	o.k.
4	450	1290	1875	345	345	3960	4	146.2	27.09	2610	o.k.
5	550	1210	1875	345	345	3980	3	147.5	26.98	3800	o.k.
6	600	1165	1875	345	345	3985	3.5	148.3	26.87	4360	o.k.

ratio for the particular combination of ingredients, needed to select proportions for a range of specified requirements.

A3.4.5 In laboratory tests, it seldom will be found, even by experienced operators, that desired adjustments will develop as smoothly as indicated in Table A3.4.1. Furthermore, it should not be expected that field results will check exactly with laboratory results. An adjustment of the selected trial mix on the job is usually necessary. Closer agreement between laboratory and field will be assured if machine mixing is employed in the laboratory. This is especially desirable if air-entraining agents are used since the type of mixer influences the amount of air entrained. Before mixing the first batch, the laboratory mixer should be "buttered" or the mix "overmortared" as described in ASTM C 192. Similarly, any processing of materials in the laboratory should simulate as closely as practicable corresponding treatment in the field.

A3.4.6 The series of tests illustrated in Table A3.4.1 may be expanded as the size and special requirements of the work warrant. Variables that may require investigation include: alternative aggregate sources, maximum sizes and gradings; different types and brands of cement; admixtures; and considerations of concrete durability, volume change, temperature rise, and thermal properties.

A3.5—Test methods

A3.5.1 In conducting laboratory tests to provide information for selecting concrete proportions, the latest revisions of the following methods should be used:

A3.5.1.1 *For tests of ingredients:*

Sampling hydraulic cement—ASTM C 183

Specific gravity of hydraulic cement—ASTM C 188

Sampling stone, slag, gravel, sand, and stone block for use as highway materials—ASTM D 75

Sieve or screen analysis of fine and coarse aggregates—ASTM C 136

Specific gravity and absorption of coarse aggregates—ASTM C 127

Specific gravity and absorption of fine aggregates—ASTM C 128

Surface moisture in fine aggregate—ASTM C 70

Total moisture content of aggregate by drying —ASTM C 566

Unit weight of aggregate—ASTM C 29

Voids in aggregate for concrete—ASTM C 30

Fineness modulus—Terms relating to concrete and concrete aggregates, ASTM C 125

A3.5.1.2 *For tests of concrete:*

Sampling fresh concrete—ASTM C 172

Air content of freshly mixed concrete by the volumetric method—ASTM C 173

Air content of freshly mixed concrete by the pressure method—ASTM C 231

Slump of portland cement concrete—ASTM C 143

Weight per cubic foot, yield, and air content (gravimetric) of concrete—ASTM C 138

Concrete compression and flexure test specimens, making and curing in the laboratory—ASTM C 192

Compressive strength of molded concrete cylinders—ASTM C 39

Flexural strength of concrete (using simple beam with third-point loading)—ASTM C 78

Flexural strength of concrete (using simple beam with center point loading)—ASTM C 293

Splitting tensile strength of molded concrete cylinders—ASTM C 496

A3.6—Mixes for small jobs

A3.6.1 For small jobs where time and personnel are not available to determine proportions in accordance with the recommended procedure, mixes in Table A3.6.1 will usually provide concrete that is amply strong and durable if the amount of water added at the mixer is never large enough to make the concrete overwet. These mixes have been predetermined in conformity with the recommended procedure by assuming conditions applicable to the average small job, and for aggregate of medium specific gravity. Three mixes are given for each maximum size of coarse aggregate. For the selected size of coarse aggregate, Mix B is intended for initial use. If this mix proves to be oversanded, change to Mix C; if it is undersanded, change to Mix A. It should be noted that the mixes listed in the table are based on dry or surface-dry sand. If the sand is moist or wet, make the corrections in batch weight prescribed in the footnote.

A3.6.2 The approximate cement content per cubic foot of concrete listed in the table will be helpful in estimating cement requirements for the job. These requirements are based on concrete that has just enough water in it to permit ready working into forms without objectionable segregation. Concrete should slide, not run, off a shovel.

TABLE A3.6.1—CONCRETE MIXES FOR SMALL JOBS

Procedure: Select the proper maximum size of aggregate (see Section 5.3.2). Use Mix B, adding just enough water to produce a workable consistency. If the concrete appears to be undersanded, change to Mix A and, if it appears oversanded, change to Mix C.

Maximum size of aggregate, in.	Mix designation	Approximate weights of solid ingredients per cu ft of concrete, lb: Cement	Sand*: Air-entrained concrete†	Sand*: Concrete without air	Coarse aggregate: Gravel or crushed stone	Coarse aggregate: Iron blast furnace slag
½	A	25	48	51	54	47
	B	25	46	49	56	49
	C	25	44	47	58	51
¾	A	23	45	49	62	54
	B	23	43	47	64	56
	C	23	41	45	66	58
1	A	22	41	45	70	61
	B	22	39	43	72	63
	C	22	37	41	74	65
1½	A	20	41	45	75	65
	B	20	39	43	77	67
	C	20	37	41	79	69
2	A	19	40	45	79	69
	B	19	38	43	81	71
	C	19	36	41	83	72

*Weights are for dry sand. If damp sand is used, increase tabulated weight of sand 2 lb and, if very wet sand is used, 4 lb.

†Air-entrained concrete should be used in all structures which will be exposed to alternate cycles of freezing and thawing. Air-entrainment can be obtained by the use of an air-entraining cement or by adding an air-entraining admixture. If an admixture is used, the amount recommended by the manufacturer will, in most cases, produce the desired air content.

APPENDIX 4— HEAVYWEIGHT CONCRETE MIX PROPORTIONING

A4.1—Concrete of normal placeability can be proportioned for densities as high as 350 lb per cu ft by using heavy aggregates such as iron ore, barite, or iron shot and iron punchings. Although each of the materials has its own special characteristics, it can be processed to meet the standard requirements for grading, soundness, cleanliness, etc. The acceptability of the aggregate should be made depending upon its intended use. In the case of radiation shielding, determination should be made of trace elements within the material which may become reactive when subjected to radiation. In the selection of materials and proportioning of heavyweight concrete, the data needed and procedures used are similar to that required for normal weight concrete except that the following items should be considered.

A4.1.1—In selecting an aggregate for a specified density, the specific gravity of the fine aggregate should be comparable to that of the coarse aggregate in order to lessen settlement of the coarse aggregate through the mortar matrix. Typical materials used as heavy aggregates include the following:

Material	Description	Specific gravity	Approx. concrete unit wt (lb/cu ft)
Limonite Goethite	Hydrous iron cres	3.4 - 3.8	180 - 195
Barite	Barium sulfate	4.0 - 4.4	205 - 225
Ilmenite Hematite Magnetite	Iron ores	4.2 - 4.8	215 - 240
Iron	Shot, pellets, punchings, etc.	6.5 - 7.5	310 - 350

A4.1.2—When the concrete in service is to be exposed to a hot, dry environment, it should be proportioned so that the fresh unit weight is at least 10 lb per cu ft higher than the required dry unit weight.

A4.1.3—When entrained air is required to resist conditions of exposure, allowance must be made for the loss in weight due to the space occupied by the air. To achieve adequate consolidation using high frequency vibrators and close insertion intervals, without the excessive loss of entrained air, plastic concrete should be designed for a high air content to offset this loss during placement.

A4.1.4—Heavyweight concrete is often used for radiation shielding. In this case, the aggregate type and concrete weight should be selected consistent with the type of radiation involved. Generally speaking the greater the mass the better are the shielding properties against gamma and beta rays. However, neutron attenuation depends more on the specific elements present in the concrete, i.e., hydrogen, carbon, boron, etc.

A4.1.5—Ferrophosphorous and ferrosilicon (heavyweight slags) materials should be used only after thorough scrutiny. Hydrogen evolution in heavyweight concrete containing these aggregates has been known to result in a reaction of a self-limiting nature, producing over 25 times its volume of hydrogen before the reaction ceases.

A4.1.6—In Section 5.3.7 (Step 7) caution must be exercised if the weight method (Section 5.3.7.1) is used to estimate the fine aggregate batch weight. The values in Table 5.3.7.1 must be corrected for overall aggregate specific gravity since the table is based on an average aggregate specific gravity of 2.7. Therefore, it is recommended that the required amount of fine aggregate be determined by the absolute volume procedure (Section 5.3.7.2).

A4.2—*Production and quality control.* The technique and equipment for producing heavyweight concrete are the same as used with normal weight concrete. In the selection of heavyweight materials and combinations thereof for the purposes of proportioning specification concretes, attention must be directed to aggregate effects on placeability, strength, and durability of the concrete. Testing and quality control measures assume greater importance than with normal weight concrete. Control of aggregate grading is essential because of the effect on the placing and consolidating properties of concrete, and on the unit weight of the concrete. In enforcing strict quality control special attention should be paid to the following:

A4.2.1—Prevention of contamination with normal weight aggregate in stockpiles and conveying equipment.

A4.2.2—Purging of all aggregate handling and batching equipment, premixers and truck mixers, before batching and mixing heavyweight concrete.

A4.2.3—Accuracy and condition of conveying and scale equipment, aggregate storage and concrete batching bins. Due to the greater weight of heavyweight aggregate, the permissible volume batched in a bin is considerably less than the design capacity. For example: a 100 ton aggregate bin designed for 75 cu yd of normal weight aggregate should not be loaded with more than 25 to 55 cu yd for the range of specific gravities shown in Section A4.1.1.

A4.2.4—Condition and loading of mixing equipment. For concrete of a weight range of approximately 4800 to 9500 lb per cu yd, the capacity of a truck mixer, without overloading, is reduced from 20 to 60 percent.

A4.2.5—Accurate aggregate proportioning to maintain w/c ratio. Degradation of some coarse heavyweight aggregates, iron ores in particular, is another production problem which should be carefully controlled. Either the coarse aggregate should be rescreened immediately prior to incorporation into the concrete or adjustments made in the mixture proportions that compensate for the increased percentage of fines in the coarse aggregate, caused by aggregate breakdown during handling. Therefore, caution should be exercised and frequent gradation checks made on stockpiled aggregates.

A4.2.6—Frequent checks of fresh unit weight.

A4.2.7—Design and construction of forms to handle additional weight of concrete.

A4.2.8—Vibrators for consolidation.

A4.3—*Example problem.* Concrete is required for counterweights on a lift bridge not subjected to freezing and thawing conditions. An average 28-day compressive strength of 4500 psi will be required. Placement conditions permit a slump of 2 to 3 in. and a maximum size aggregate of 1 in. The design of the counterweight requires a dry unit weight of 230 lb per cu ft. An investigation of economically available materials has indicated the following:

Cement	—Type I (non-air-entraining)
Fine aggregate	—Specular Hematite
Coarse aggregate	—Ilmenite

The table in Section 4.1.1 indicates that this combination of materials may result in a dry unit weight of 215 to 240 lb per cu ft. The following properties of the aggregates have been obtained from laboratory tests:

	Fine aggregate	Coarse aggregate
Fineness modulus	2.30	—
Specific gravity (Bulk SSD)	4.95	4.61
Absorption (percent)	0.05	0.08
Dry rodded weight	—	165 lb per cu ft
Maximum size	—	1 in.

Employing the sequence outlined in Section 5 of this recommended practice, the quantities of ingredients per cubic yard of concrete are calculated as follows:

A4.3.1 *Step 1.* As indicated above, the desired slump is 2 to 3 in.

A4.3.2 *Step 2.* The available aggregate sources have been indicated as suitable, and the coarse aggregate will be a well-graded and well-shaped crushed ilmenite with a maximum size of 1 in.

A4.3.3 *Step 3.* By interpolation in Table 5.3.3, non-air-entrained concrete with a 2 to 3 in. slump and a 1 in. maximum size aggregate requires a water content of approximately 310 lb per cu yd. The estimated entrapped air is 1.5 percent. (Non-air-entrained concrete will be used because (1) the concrete is not exposed to severe weather, and (2) a high air content could reduce the dry unit weight of the concrete.)

Note: Table 5.3.3 values for water requirement are based on the use of well-shaped crushed coarse aggregates. Void content of compacted dry fine or coarse aggregate can be used as an indicator of angularity. Void contents of compacted 1 in. coarse aggregate of significantly more than 40 percent indicate angular material which will probably require more water than that listed in Table 5.3.3. Conversely rounded aggregates with voids below 35 percent will probably need less water.

A4.3.4 *Step 4.* From Table 5.3.4(a) the water-cement ratio needed to produce a strength of 4500 psi in non-air-entrained concrete is found to be approximately 0.52.

A4.3.5 *Step 5.* From the information derived in Steps 3 and 4, the required cement content is calculated to be $310/0.52 = 596$ lb per cu yd.

A4.3.6 *Step 6.* The quantity of coarse aggregate is estimated by extrapolation from Table 5.3.6. For a fine aggregate having a fineness modulus of 2.30 and a 1 in. maximum size aggregate, the table indicates that 0.72 cu ft of coarse aggregate, on a dry-rodded basis, may be used in each cubic foot of concrete. For a cubic yard, therefore, the coarse aggregate will be $27 \times 0.72 = 19.44$ cu ft. Since the dry-rodded unit weight of the coarse aggregate is 165 lb per cu ft, the dry weight of coarse aggregate to be used in a cubic yard of concrete would be $19.44 \times 165 = 3208$ lb. The angularity of the coarse aggregate is compensated for in the ACI proportioning method through the use of the dry-rodded unit weight; however, the use of an extremely angular fine aggregate may require a higher proportion of fine aggregate, an increased cement content, or the use of air-entrainment to produce the required workability.

A4.3.7 *Step 7.* For heavyweight concrete, it is recommended that the required fine aggregate be determined on the absolute volume basis. With the quantities of cement, water, air and coarse aggregate established, the sand content can be calculated as follows:

Volume of water	$= \dfrac{310}{62.4}$	= 4.97 cu ft
Volume of air	$= 0.015 \times 27$	= 0.40 cu ft
Solid volume of cement	$= \dfrac{596}{3.15 \times 62.4}$	= 3.03 cu ft
Solid volume of coarse aggregate	$= \dfrac{3208}{4.61 \times 62.4}$	= 11.15 cu ft
Total volume of all ingredients except sand		= 19.55 cu ft
Solid volume of sand	$= 27 - 19.55$	= 7.45 cu ft
Required weight of sand	$= 7.45 \times 4.95 \times 62.4$	= 2301 lb

A4.3.8 *Step 8.* Tests indicate total moisture of 0.15 percent in the fine aggregate and 0.10 percent in the coarse aggregate; therefore, the adjusted aggregate weights become:

Fine aggregate (wet)	$= 1.0015 \times 2301 = 2304$ lb
Coarse aggregate (wet)	$= 1.0010 \times 3208 = 3211$ lb

Absorbed water does not become part of the mixing water and must be excluded from the adjustment in added water. Thus surface water contributed by the fine aggregate amounts to $0.15 - 0.05 = 0.10$ percent; by the coarse aggregate $0.10 - 0.08 = 0.02$ percent. The estimated requirement for added water, therefore becomes:

$$310 - 2301(0.001) - 3208(0.0002) = 307 \text{ lb}$$

A4.3.9 *Step 9.* The resulting estimated proportions by weight of the heavyweight concrete becomes:

Cement	= 596 lb
Fine aggregate (wet)	= 2304 lb
Coarse aggregate (wet)	= 3211 lb
Water	= 307 lb
Estimated unit wt (fresh)	= 6418/27 = 237.7 lb per cu ft

A4.4—The above heavyweight concrete proportioned mixture was actually used for approximately 5060 cu yd. Field adjustments resulted in the following actual batch weights:

Cement	590 lb
Fine aggregate	2310 lb
Coarse aggregate	3220 lb
Water	285 lb (plus a water-reducing agent)

The actual field test results indicated the concrete possessed the following properties:

Unit weight (fresh)	235.7 lb per cu ft
Air content	2.8 percent
Slump	2½ in.
Strength	5000 psi at 28 days

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ACI Standard

Recommended Practice for Selecting Proportions for Structural Lightweight Concrete (ACI 211.2-69)*

(Revised 1977)

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Describes, with examples, a method for proportioning and adjusting structural grade concrete containing lightweight aggregates. The method described uses a "specific gravity factor," determined by pycnometer test on the aggregates, which accounts for variations in moisture content of the aggregates. A tabular form is suggested for systematic calculation of batch weights and "effective displaced volumes." Examples are given for adjustments for change in aggregate moisture content, aggregate proportions, cement factor, slump, and air content.

Keywords: absorption; aggregate gradation; air content; air entrainment; calibration; cement content; coarse aggregates; fine aggregates; fineness modulus; lightweight aggregate concretes; lightweight aggregates; mix proportioning; moisture; sampling; slump tests; specific gravity; testing; water.

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†Members of Subcommittee 1 which prepared this report.

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CHAPTER 1—INTRODUCTION

1.1—Purpose

The purpose of this standard is to provide a generally applicable method for selecting and adjusting mix proportions for structural lightweight concrete.[1]

Other procedures, such as those used for proportioning normal weight concrete,[2,3] may be used with some lightweight aggregates, particularly with those having a low rate of absorption and a low total absorption. This standard provides a proportioning method which is applicable to most structural lightweight aggregate concretes, and is also applicable to normal weight concretes, and to concretes containing a combination of lightweight and normal weight aggregates.[4,5,6]

1.2—Scope

Discussion in this standard is limited to structural grade lightweight aggregates and structural lightweight aggregate concretes.[7,8] Structural lightweight aggregate concrete is defined as concrete which: (a) is made with lightweight aggregates conforming to ASTM C 330, (b) has a compressive strength in excess of 2500 psi (175 kg/cm^2) at 28 days when tested in accordance with methods stated in ASTM C 330, and (c) has an air dry weight not in excess of 115 lb per cu ft (1840 kg/m^3) as determined by ASTM C 567. Concrete in which a portion of the lightweight aggregates is replaced by normal weight aggregates, and which meets the strength and weight limitations noted, is within the scope of this standard.[9] Normal weight aggregates where used should conform to the requirements of ASTM C 33.

1.3—Specifications cited in this standard

The specifications of the American Society for Testing and Materials referred to in this report are listed below with their serial designation including the year of adoption or revision: *

C 33-74	Standard Specification for Concrete Aggregates
C 127-73	Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate
C 128-73	Standard Method of Test for Specific Gravity and Absorption of Fine Aggregate
C 173-73	Standard Method of Test and Air Content of Freshly Mixed Concrete by the Volumetric Method
C 330-69	Standard Specification for Lightweight Aggregates for Structural Concrete
C 567-71	Standard Method of Test for Unit Weight of Structural Lightweight Concrete

*The specifications listed were the latest at the time this standard was prepared. Since these specifications are revised frequently, generally in minor details only, the user of this standard should check directly with the sponsoring society if it is desired to refer to the latest edition.

CHAPTER 2—FACTORS AFFECTING PROPORTIONING OF LIGHTWEIGHT AGGREGATE CONCRETE

2.1—Aggregates—Absorption and moisture content[1]

2.1.1—The principal factors necessitating modification of proportioning and control procedures for lightweight aggregate concrete, compared to normal weight concrete, are the greater absorption value and the higher rate of absorption of most lightweight aggregates. The greater absorption is due to the cellular nature of lightweight aggregates.

2.1.2—The moisture content of a lightweight aggregate at some time after it comes into contact with water depends on both the time of exposure to water and on the initial moisture content of the aggregate. Initially damp lightweight aggregates usually contain more total absorbed water after a short period of water immersion than is contained in similar, initially dry aggregates after immersion for the same period. Thus, concretes made with initially dry lightweight aggregates contain less total water than similar concretes made with similar aggregates that were damp for some time before mixing.[1,10,11]

2.1.3—Moisture content of the aggregates at time of mixing has little effect on compressive strength of the concrete, providing the moisture content is known and the mix is adjusted to provide constant cement and air contents, similar consistency, and a constant volume of both coarse and fine aggregates on the dry basis.[1,10,12]

2.1.4—Concrete made with dry lightweight aggregates will have lower fresh unit weight than that made with initially damp aggregates, but weights at later ages after normal drying tend to equalize.[1]

2.1.5—Damp aggregates are usually preferable to dry aggregates at time of mixing. Damp aggregates have less tendency to segregate in storage. Damp aggregates absorb less water during mixing and thus reduce the possibility of loss in slump as the concrete is being mixed, transported, and placed.

2.1.6—When concrete is made with lightweight aggregates having a low initial moisture content (usually less than 8-10 percent) and a relatively high rate of absorption, it may be desirable to mix the aggregates with one-half to two-thirds of the mixing water for a short period prior to the addition of cement and air-entraining agent in order to minimize slump loss.[5,13] The supplier of the particular aggregate should be consulted regarding the necessity for such predamping and for mixing procedure generally.

2.1.7—Many lightweight aggregates that contain some absorbed moisture, but which are not saturated at the time of mixing, have a very low rate of continued absorption. In this condition, little absorption occurs during mixing and placing so that predampening is not necessary, and negligible slump loss occurs.[11]

2.1.8—Concrete made with saturated lightweight aggregates may be more vulnerable to freezing and thawing than concrete made with damp or dry lightweight aggregates, unless the concrete is allowed to lose its excess moisture, after curing, prior to such exposure.[14,15]

2.2—Aggregates—Gradation

2.2.1—Grading of the fine and coarse aggregates and the proportions used have an important effect on the concrete.[11] A well graded aggregate will have a minimum void content and so will require a minimum amount of paste to fill the voids. This will usually result in the most economical use of cement and will provide maximum strength with minimum volume change due to drying shrinkage and temperature changes.

2.2.2—For normal weight aggregates the bulk specific gravities of fractions retained on the dif-

TABLE 2.2.2—COMPARISON OF FINENESS MODULUS BY WEIGHT AND BY VOLUME FOR A TYPICAL LIGHTWEIGHT AGGREGATE

Sieve size	Opening		Percent retained by weight	Cumulative percent retained by weight	Bulk specific gravity s.s.d. basis	Percent retained by volume	Cumulative percent retained by volume
	in.	mm					
4	0.187	4.76	0	. . .	1.40	—	—
8	0.0937	2.38	21.6	21.6	1.55	25.9	25.9
16	0.0469	1.19	24.4	46.0	1.78	25.4	51.3
30	0.0232	0.590	18.9	64.9	1.90	18.5	69.8
50	0.0117	0.297	14.0	78.9	2.01	12.9	82.7
100	0.0059	0.149	11.6	90.5	2.16	10.0	92.7
Pan			9.5	100.0	2.40	7.3	100.0

Fineness modulus (by weight) = 3.02 Fineness modulus (by volume) = 3.22

ferent sieve sizes are nearly equal. Percentages retained on each size indicated by weight, therefore, give a true indication also of percentages by volume. However, the bulk specific gravity of the various size fractions of lightweight aggregate increases as the particle size decreases. Some coarse aggregate particles may float on water, whereas material passing the No. 100 sieve (0.15 mm) may have a specific gravity approaching that of natural sand. It is the volume occupied by each size fraction, and not the weight of material retained on each sieve, that determines the void content and paste content, and influences workability of the concrete. Percentages retained on each sieve and fineness modulus, by weight and by volume, are computed for comparison in the example of Table 2.2.2.

Fineness modulus of 3.22 by volume in the example indicates a considerably coarser grading than that normally associated with the fineness modulus of 3.02 by weight. Therefore, lightweight aggregates require a larger percentage of material retained on the finer sieve sizes, on a weight basis, than do normal weight aggregates, to provide an equal size distribution by volume.

2.2.3—Grading limitations and other properties necessary for lightweight aggregate for structural concrete are specified in ASTM C 330.

2.2.4—As indicated in Section 1.2, concrete containing some normal weight aggregates, e.g., natural sand, is classed as lightweight concrete provided the strength and weight requirements are met. The use of natural sand usually results in some increase in strength and modulus of elasticity; these increases, however, are made at the sacrifice of increased weight. Mix proportions selected, therefore, should consider these properties in conjunction with the corresponding effects on over-all economy of the structure. (See also Section 3.2 and footnote to Section 4.3.1).

2.3—Water-cement ratio

2.3.1—The net water-cement ratio of most lightweight concrete mixes cannot be established with sufficient accuracy to use as a basis for mix proportioning.[16] This is due to the difficulty in determining how much of the total water is absorbed in the aggregate and, therefore, not available for reaction with the cement while the concrete is in its plastic state. Lightweight aggregate concrete mixes are usually established by trial mixes proportioned on a cement and air content basis at the required consistency.

2.4—Air entrainment

2.4.1—Air entrainment is desirable in most lightweight aggregate concrete as it is in most normal weight concrete. It enhances workability,[13,15,16,17] it improves resistance to freezing and thawing cycles[18] and to deicer chemicals, it decreases bleeding, and it tends to obscure minor grading deficiencies.[15]

2.4.2—The strength of concrete mixes may be reduced by air entrainment. However, at normal air contents (about 6 percent) if the cement factor of the mix is maintained and water content is reduced to maintain slump, the reduction is usually not great. Strength reduction is more apparent with richer mixes, but with lean or harsh mixes air entrainment may result in a strength increase.

2.4.3—It is recommended that lightweight aggregate concretes which may be subjected to freezing and thawing, or to deicer salts, should contain 4 to 8 percent air when maximum aggregate size is ¾ in. (20 mm), and 5 to 9 percent when maximum aggregate size is ⅜ in. (10 mm).

2.4.4—The volumetric method of measuring air, as described in ASTM C 173, gives the most reliable method of measuring air in either air-entrained or non-air-entrained lightweight concrete, and is recommended.

CHAPTER 3—ESTIMATING FIRST TRIAL MIX PROPORTIONS

3.1—General

The best approach to making a first trial mix of lightweight concrete to have given properties, and using a particular lightweight aggregate source, is to use proportions previously established for similar concrete using the same aggregate source.

Such proportions may be obtained from the aggregate supplier and may be the result of either laboratory mixes or of actual mixes supplied to jobs. These mixes may then be adjusted as necessary to change the properties or proportions using the methods described in Chapter 4. The purpose of Chapter 3 is to provide a guide to proportioning a first trial mix where such prior information is not available, following which the adjustment procedures of Chapter 4 may be used.

3.2—Volumes and proportions of aggregates

The total volume of aggregates required, measured as the sum of the uncombined volumes on a dry-loose basis, will usually be from 28 to 32 cu ft per cu yd (1.04 to 1.19 m^3/m^3). Of this amount, the fine aggregate may be from 40 to 60 percent. Both the total volume of aggregate required and the proportions of fine and coarse aggregates are dependent on several variables; these variables relate to both the nature of the aggregates and to the properties of the concrete to be produced.

In general, the least total volume of aggregates is required (a) when both the coarse and fine aggregates are well graded from the largest to the smallest sizes, (b) when the particle shape is from rounded to cubical in shape, and (c) when the surface texture is least porous. Conversely, concrete containing aggregates which tend to be angular in shape, more porous in surface texture, and possibly deficient in one or more particle sizes, will require the greater volume of aggregates. These same factors of grading, particle shape and texture also affect the percentage of fine aggregate required, a minimum percentage of fine aggregate being associated with the rounded to cubical shape, the less porous textures and the more suitable gradations. It is usual that when a well graded natural sand is used to replace lightweight fine aggregates, the proportion of coarse lightweight aggregates may be increased. The proportion of coarse aggregate should approach the maximum, consistent with workability and placeability unless tests indicate that a lesser proportion provides optimum strength. In some cases, strength ceiling may be increased by reducing the maximum size of aggregate.[7]

The effective displaced volume of aggregates (see Section 4.1.) makes up the difference between the solid volume of concrete required (e.g., 1 cu yd or 1 m^3), and the effective displaced volumes of the cement, air, and water. It follows that some reduction in volume of aggregate is required (usually in the fine fraction) with greater cement factors, greater air contents, and increased water. In an initial trial mix, therefore, an improved estimate of volume of aggregates and proportions of fine and coarse aggregates may be attained by a prior general assessment of the foregoing points. The proportions of coarse and fine aggregate will have a considerable bearing on workability and finishability of the plastic concrete; operator judgment and "feel" is, therefore, an essential element in evaluating the trial mix.

3.3—Estimating cement content

In well proportioned mixes, the cement content-strength relation is constant for given aggregates but may vary widely from one lightweight aggregate type and source to another.[16, 17] Fig. 3.3 gives an approximation of this range. It is recommended that the aggregate producer should be consulted to obtain a closer approximation of cement content required to achieve desired strength with the specific aggregate. When this information is not available, the only alternative is to make a sufficient number of trial mixes with varying cement contents such as to achieve a range of compressive strengths including the compressive strength desired.

3.4—Water

As with normal weight concrete, the amount of water which should be added to a lightweight concrete mix is the minimum amount which will permit the concrete to be properly placed, consolidated, and finished. Excess water increases the possibility of segregation, lowers strength, reduces durability, increases shrinkage, and hampers finishing. In most conditions, slumps less than 4 in. (10 cm) will be satisfactory. The effect of absorbed water is discussed in Chapter 4.

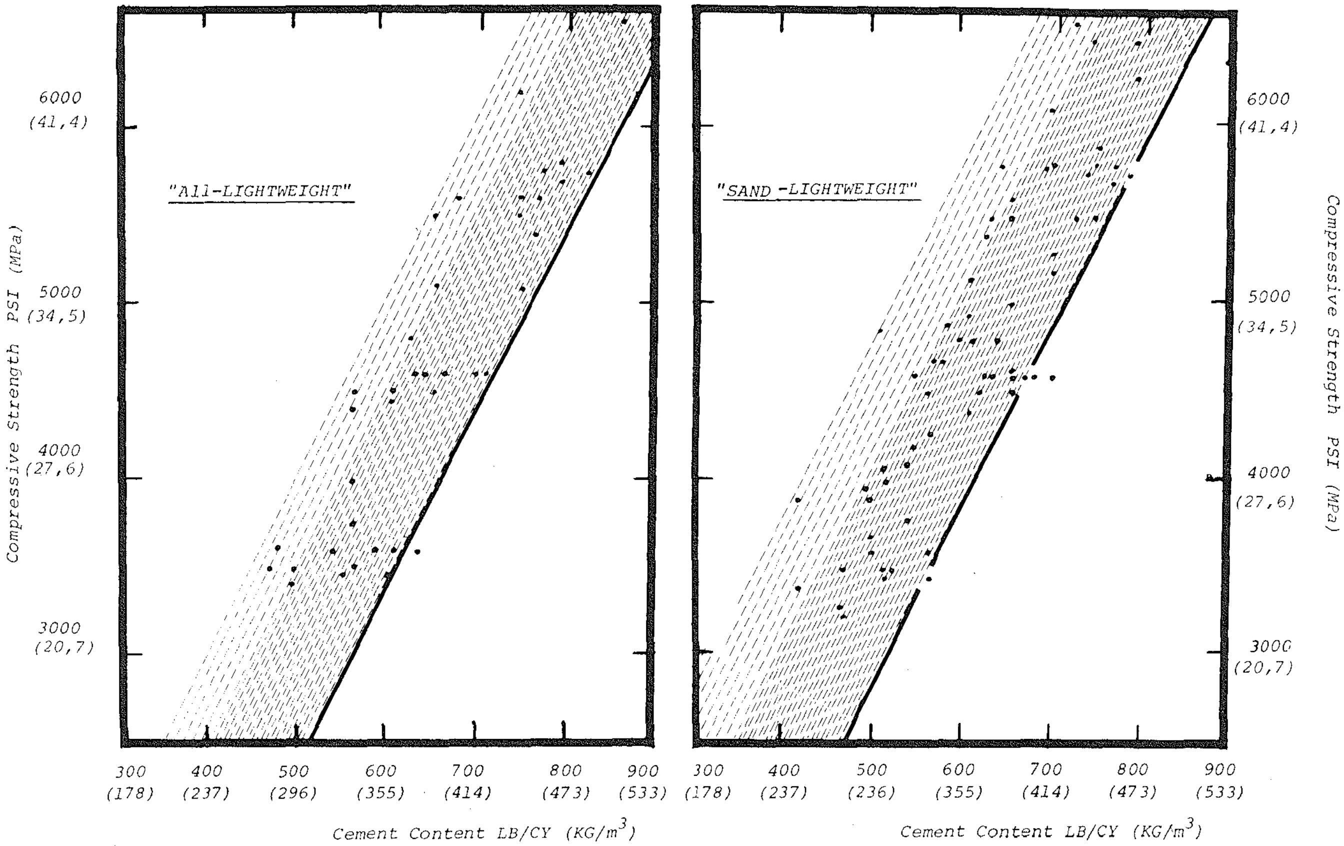

Fig. 3.3—Relationship of compressive strength and cement content of field concrete for "all-lightweight" and "sand-lightweight" aggregate

3.5—First trial mix

3.5.1—Assume a mix is required to contain 600 lb of cement per cu yd of concrete (356 kg/m³), that the dry-loose unit weights for the coarse and fine aggregates have been determined as 45 and 56 lb per cu ft (720 and 900 kg/m³), respectively, and that the moisture contents of the aggregates at time of mixing are 3 and 8 percent for coarse and fine aggregates, respectively.

Assume that the total volume of aggregate required (sum of uncombined coarse and fine aggregates, dry-loose basis) is 32 cu ft per cu yd (1.2 m³/m³), and that they are to be proportioned in equal volumes (See Section 3.2). Table 3.5.1a shows the assumed results of the first trial mix based on an attempted batch volume of 1 cu ft (Table 3.5.1b for metric units, based on attempted batch volume of 0.03 m³). Assume 13.5 lb (6.2 kg) of added water is necessary to produce the required slump. Note that while the actual mix is made with damp aggregates, the calculated aggregate weights are shown here on the dry basis only, and the weight of water shown is total water including both added water and the water contained (absorbed) in the aggregates.

TABLE 3.5.1a—FIRST TRIAL BATCH

	Batch weights dry basis, lb	Cu yd weights dry basis, lb
Cement	$\frac{600}{27} = 22.2$	26.06 × 22.2 = 579
Coarse aggregate	$\frac{16 \times 45}{27} = 26.7$	26.06 × 26.7 = 696
Fine aggregate	$\frac{16 \times 56}{27} = 33.2$	26.06 × 33.2 = 865
Total water	17.0	26.06 × 17.0 = 443
Total weight	99.1	2583

TABLE 3.5.1b—FIRST TRIAL BATCH (METRIC UNITS)

	Batch weights dry basis, kg	Weights per m³ dry basis, kg
Cement	0.03 × 356 = 10.7	32.06 × 10.7 = 343
Coarse aggregate	$0.03 \times \frac{1.2}{2} \times 720 = 13.0$	32.06 × 13.0 = 417
Fine aggregate	$0.03 \times \frac{1.2}{2} \times 900 = 16.2$	32.06 × 16.2 = 519
Total water	7.9	32.06 × 7.9 = 253
Total weight	47.8	1532

TABLE 3.5.2a—ADJUSTMENT TO FIRST TRIAL BATCH

	First trial mix		Adjusted mix	
	Dry basis, lb	Damp basis, lb	Dry basis, lb	Damp basis, lb
Cement	579	579	579 + 21 = 600	579 + 21 = 600
Coarse aggregate	696	1.03 × 696 = 717	696 − 5 = 691	1.03 × 691 = 712
Fine aggregate	865	1.08 × 865 = 934	865 − 6 = 859	1.08 × 859 = 928
Water*	443	443 − 90 = 353	443	443 − 90 = 353
Total	2583	2583	2593	2593

*Water is total water in dry basis tabulations; added water in damp basis tabulations.

TABLE 3.5.2b—ADJUSTMENT TO FIRST TRIAL BATCH (METRIC UNITS)

	First trial mix		Adjusted mix	
	Dry basis, kg	Damp basis, kg	Dry basis, kg	Damp basis, kg
Cement	343	343	343 + 13 = 356	356
Coarse aggregate	417	1.03 × 417 = 430	417 − 3 = 414	1.03 × 414 = 426
Fine aggregate	519	1.08 × 519 = 561	519 − 4 = 515	1.08 × 515 = 556
Water*	253	253 − 55 = 198	253	253 − 53 = 200
Total	1532	1532	1538	1538

*Water is total water in dry basis tabulations; added water in damp basis tabulations.

Fresh unit weight is determined by test as 95.7 lb per cu ft (1533 kg/m^3). Actual yield is calculated as 99.1/95.7 = 1.036 cu ft (47.8/1533 = 0.0312 m^3). Quantities per cu yd (per m^3) are then obtained by multiplying batch quantities by 27/1.036 = 26.06 (in metric units, 1.000/0.0312 = 32.06) as shown in Table 3.5.1a (Table 3.5.1b for metric units). An air test is made on the trial batch to determine if it is within desired limits.

3.5.2—The cement factor of the first trial mix is seen to be 21 lb per cu yd (13 kg/m^3) less than was desired. If other characteristics of the mix are satisfactory, the quantities can be adjusted on the basis of experience or "rules of thumb." For example, it may be assumed that the volume of aggregate on a dry-loose basis must be changed about 0.01 cu ft in the opposite direction for each 1 lb change in cement (about 0.0006 m^3 for each 1 kg change in cement). To increase cement by 21 lb (13 kg) in the example, therefore, it is necessary to decrease the aggregate quantity by 0.01 × 21 = 0.21 cu ft (0.0006 × 13 = 0.008 m^3). Allocating the reduction equally to coarse and fine aggregates, reductions (on the dry basis) are:

coarse aggregate

$$\frac{0.21}{2} \times 45 = 5\text{ lb} \qquad \frac{0.008}{2} \times 720 = 3\text{ kg}$$

fine aggregate

$$\frac{0.21}{2} \times 56 = 6\text{ lb} \qquad \frac{0.008}{2} \times 900 = 4\text{ kg}$$

For these small adjustments, the required amount of added water will not be changed appreciably. The correction of weights is shown in Table 3.5.2a (Table 3.5.2b for metric units) on both the dry and damp basis. Note that it is convenient to tabulate the mix on the dry basis for future adjustments.

It will be noted in Tables 3.5.2a or 3.5.2b that total water was assumed to remain unchanged between the dry basis and the damp basis. This assumption may be made for purposes of rough adjustments but, as will be shown in Chapter 4, total water will actually change with changes in the moisture condition of the aggregates at time of mixing.[1] For rough adjustments to the mix in Tables 3.5.2a or 3.5.2b for changes in moisture content of the aggregate, the dry basis aggregate weights are simply multiplied by the appropriate moisture percentages, and the weight of water thus added to the aggregate weights is deducted from the added water. For more accurate adjustments, specific gravity factor data should be obtained, and corrections made as shown in Chapter 4, Section 4.3.

3.6—Weight method

Lightweight aggregate concrete may also be proportioned by the weight method. This method utilizes the fact that the sum of the weights of all ingredients in a mix is equal to total weight of the same mix. If the weight of the particular concrete per unit volume, and containing a particular aggregate, can be estimated, and if the weight of the cement and water in the same unit volume is known or can be estimated, the weight of the lightweight aggregates in that volume can be determined by subtraction.

CHAPTER 4—ADJUSTING MIX PROPORTIONS

4.1—General

The method described herein for proportioning and/or adjusting lightweight concrete mixes is a variation on the method of absolute volumes. In the method of absolute volumes, the volume displaced or "occupied" by each ingredient of the mix (except entrained air) is calculated as the weight in lb (kg) of that ingredient divided by the product of 62.4 lb per cu ft (1000 kg/m^3) and the specific gravity of that ingredient. Total volume of the mix is the sum of the displaced or absolute volume of each ingredient thus calculated plus the volume of entrained and entrapped air determined by direct test.

Calculation of the absolute volume of cement, based on dry weight of cement in the mix, and calculation of air as the percentage air determined by test multiplied by total volume of mix, are the same for both lightweight concrete and normal weight concrete. The volume displaced by normal weight aggregates is calculated on the basis of the saturated surface dry weights of aggregates and the bulk specific gravities (saturated surface dry basis) as determined by ASTM C 127 and C 128. Volume displaced by water in normal weight concrete mixes is, therefore, on the basis of "net" mix water. Net mix water is the water added at the mixer plus any surface water on the aggregates or minus any water absorbed by aggregates which are less than saturated.

The effective volume displaced by lightweight aggregates in concrete is calculated on the basis of weights of aggregates with a known moisture content as used, and on a specific gravity factor[4,6] which is a function of the moisture content of the aggregate, and which is determined as described in Appendix A. Effective displaced volume of water in lightweight concrete mixes is then based on the actual water added at the mixer.

4.2—Specific gravity factor (pycnometer method)*[3]

4.2.1—The relationship of weight to displaced volume for lightweight aggregates, as determined

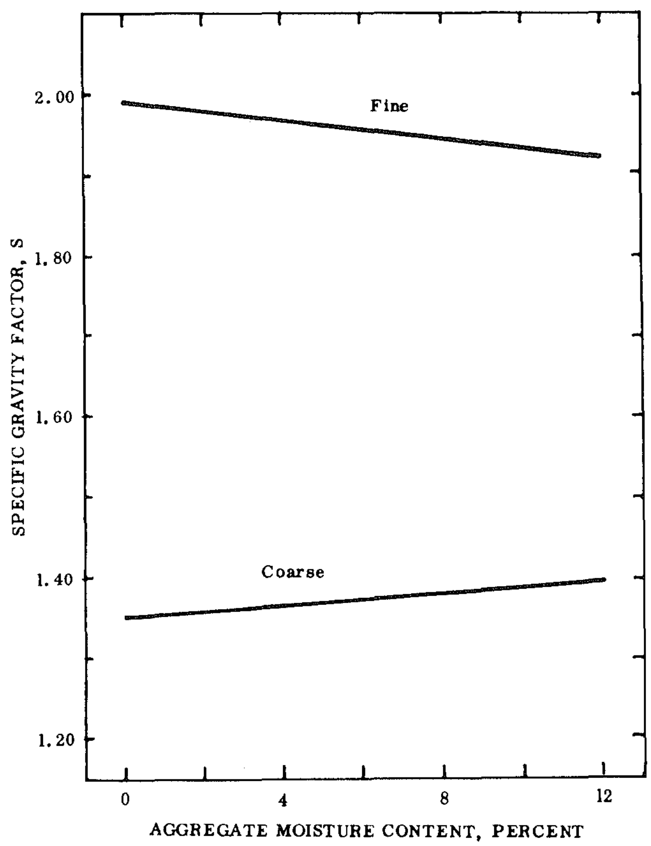

Fig. 4.2.2—Example of the relationship between pycnometer specific gravity factor and moisture content for a lightweight aggregate

*The specific gravity factor described and used herein is not the same weight-volume relation, nor are the numerical values the same, as described in ACI 613A-59.

by the method of Appendix A, is termed a specific gravity factor.[1] It is the ratio of the weight of the aggregates as introduced into the mixer, to the effective volume displaced by the aggregates. The weight of aggregates as introduced into the mixer includes any moisture absorbed in the aggregate and any free water on the aggregates.

4.2.2—Specific gravity factors generally vary with moisture content of aggregates.[1] For each aggregate type and gradation, therefore, it is necessary to determine by the method of Appendix A the specific gravity factors over the full range of moisture conditions likely to be encountered in service. Fig. 4.2.2 shows a typical plot of such determinations. The variation is usually approximately linear in the lower range of moisture contents, but may digress from linearity at higher moisture contents. The full curve, therefore, should be established and extrapolation should be avoided.

4.2.3—Indicated specific gravity factors of aggregates increase slightly with time of immersion in the pycnometer because of continued aggregate absorption. The rate of increase becomes smaller with longer immersion periods. The increase with time of immersion generally is greatest when the aggregate is tested in the dry condition and will become smaller as the moisture content of the aggregate before immersion increases. Pycnometer specific gravity factors obtained after 10 min immersion of aggregates should normally be suitable for mix proportioning and adjustment procedures. Where some loss of slump is anticipated in long haul ready-mixed concrete operations due to continued absorption of water into the aggregates, additional water is required to offset the resultant loss of yield. The mix proportions should be determined on the basis of the 10 min specific gravity factor. However, a calculation of the lower effective displaced volumes of aggregates, based on the longer time specific gravity factor, should provide guidance to the anticipated loss of yield to be compensated for by additional water.

4.3—Examples of adjustment procedures

4.3.1—Both field mixes and laboratory mixes may require adjustment from time to time either to compensate for some unintentional change in the characteristics of the concrete or to make a planned change in the characteristics. Adjustment may be required, for example, to compensate for a change in moisture content of the aggregates; it may be desired to proportion a mix for greater or lesser cement content; or perhaps, a change in slump or air content may be necessary. These adjustments can be made with considerable confidence based on either a first trial mix as described in Chapter 3, or on previous field or laboratory mixes with similar aggregates. Small mixes of perhaps 1.0 to 2.0 cu ft (0.03 to 0.06 m^3) total volume which are made and adjusted in the laboratory will require some further adjustments when extrapolated to field mixes of possibly 100 to 300 times the laboratory volume. It is recommended that tests of fresh unit weight, air content, and slump be made on the initial field mixes, and any necessary adjustments be made on the field batch quantities. Procedures for adjustments are illustrated in Sections 4.3.3 through 4.3.7.*

4.3.2 *Guides for adjusting mixes*—When it is desired to change the amount of cement, the volume of air, or the percentage of fine aggregate in a mix, or when it is desired to change the slump of the concrete, it is necessary to offset such changes with adjustments in one or more other factors, if yield and other characteristics of the concrete are to remain constant. The following paragraphs indicate some of the compensating adjustments, show the usual direction of adjustments necessary, and give a rough approximation of the amount of the adjustments per cu yd (m^3) of concrete. It should be noted, however, that the numerical values given are intended for *guidance only*, that they are *approximations*, and that more accurate values obtained by observation and experience with particular materials should be used wherever possible.

4.3.2.1 Proportion of fine aggregate. An increase in the percentage of fine to total aggregates generally requires an increase in water content. For each 1 percent increase in fine aggregate, increase water by approximately 3 lb per cu yd (2 kg/m^3). Increase in water content will require an increase in cement content to maintain strength. For each 3 lb per cu yd (2 kg/m^3) increase in water, increase cement by approximately 1 percent. Adjustment should be made in the coarse and fine aggregate weights as necessary to obtain desired proportions of each, and to maintain required total effective displaced volume. An example of adjustment for change in fine aggregate proportion is shown in Section 4.3.4.

*The examples of adjustment procedures shown in Sections 4.3.3 through 4.3.7 assume the use of all lightweight aggregates. If part or all of the lightweight fine aggregate is replaced by normal weight sand, it is recommended that the adjustments be based on an original mix which incorporates normal weight sand, with adjustments to the natural sand fraction being based either (1) on saturated surface dry weight and bulk specific gravity, saturated surface dry basis or, (2) on aggregate in moisture condition as used and a corresponding specific gravity factor.

4.3.2.2 Air content. An increase in air content will be accompanied by an increase in slump unless water is reduced to compensate. For each 1 percent increase in air content, water should be decreased by approximately 5 lb per cu yd (3 kg/m^3). An increase in air content may be accompanied by a decrease in strength unless compensated for by additional cement. (See Section 2.4.2). Adjustment should be made in fine aggregate weight as necessary to maintain required total effective displaced volume. An example of adjustment for change in air content is shown in Section 4.3.6.

4.3.2.3 Slump. An increase in slump is obtained by increasing water content. For each desired 1 in. (25 mm) increase in slump, water should be increased approximately 10 lb per cu yd (6 kg/m^3) when initial slump is about 3 in. (75 mm); somewhat more when initial slump is lower; somewhat less when intial slump is higher. Increase in water content will be accompanied by a decrease in strength unless compensated for by an increase in cement content. For each 10 lb per cu yd (6 kg/m^3) increase in water, increase cement content approximately 3 percent. Adjustment should be made in fine aggregate weight as necessary to maintain required total effective displaced volume. An example of adjustment for change in slump is shown in Section 4.3.7.

4.3.3 *Adjustment for changes in aggregate moisture condition*—Assume that a field mix consists of the batch weights of materials per cu yd (per m^3), based on aggregates in the damp condition, shown in upper right quadrant of Table 4.3.3a (Table 4.3.3b for metric units), and that this mix has satisfactory characteristics for the particular application. Assume that moisture contents of the coarse and fine aggregates were m_c = 1½ percent* and m_f = 4 percent*, respec-

*In the adjustment tables the following notation applies:
m = moisture content of aggregate as percent of dry weight. Subscript c refers to coarse aggregate; subscript f refers to fine aggregate
S = specific gravity factor. Numerical subscripts refer to moisture percentage.

TABLE 4.3.3a—ADJUSTMENT FOR CHANGES IN AGGREGATE MOISTURE CONTENT

	Original mix dry basis		Original mix damp basis	
	m_c = 0 percent, S_{c0} = 1.34	m_f = 0 percent, S_{f0} = 1.99	m_c = 1½ percent, $S_{c1½}$ = 1.35	m_f = 4 percent, S_{f4} = 1.97
	Weight, lb	Effective displaced volume, cu ft	Weight, lb	Effective displaced volume, cu ft
Cement	700	3.56	700	$\frac{700}{62.4 \times 3.15}$ = 3.56
Air 5½ percent	—	1.49	—	27 × 0.055 = 1.49
Coarse aggregate	$\frac{600}{1.015}$ = 591	$\frac{591}{62.4 \times 1.34}$ = 7.07	600	$\frac{600}{62.4 \times 1.35}$ = 7.12
Fine aggregate	$\frac{963}{1.04}$ = 926	$\frac{926}{62.4 \times 1.99}$ = 7.46	963	$\frac{963}{62.4 \times 1.97}$ = 7.83
Added water	62.4 × 7.42 = 463	27.00 − 19.58 = 7.42	437	$\frac{437}{62.4}$ = 7.00
Total	2680	27.00	2700	27.00
	Adjusted mix dry basis*		Adjusted mix damp basis	
	m_c = 0 percent, S_{c0} = 1.34	m_f = 0 percent, S_{f0} = 1.99	m_c = 5 percent, S_{c5} = 1.37	m_f = 8 percent, S_{f8} = 1.95
	Weight, lb	Effective displaced volume, cu ft	Weight, lb	Effective displaced volume, cu ft
Cement	700	3.56	700	3.56
Air 5½ percent	—	1.49	—	1.49
Coarse aggregate	591	7.07	591 × 1.05 = 621	$\frac{621}{62.4 \times 1.37}$ = 7.26
Fine aggregate	926	7.46	926 × 1.08 = 1000	$\frac{1000}{62.4 \times 1.95}$ = 8.22
Added water	463	7.42	62.4 × 6.47 = 404	27.00 − 20.53 = 6.47
Total	2680	27.00	2725	27.00

*When adjustment is made only to account for changes in aggregate moisture content, the adjusted mix, dry basis, is necessarily the same as original mix, dry basis.

TABLE 4.3.3b—ADJUSTMENT FOR CHANGES IN AGGREGATE MOISTURE CONTENT (METRIC UNITS)

	Original mix dry basis		Original mix damp basis	
	$m_c = 0$ percent, $S_{c0} = 1.34$	$m_f = 0$ percent, $S_{f0} = 1.99$	$m_c = 1\frac{1}{2}$ percent, $S_{c1\frac{1}{2}} = 1.35$	$m_f = 4$ percent, $S_{f4} = 1.97$
	Weight, kg	Effective displaced volume, m^3	Weight, kg	Effective displaced volume, m^3
Cement	415	0.132	415	$\frac{415}{1000 \times 3.15} = 0.132$
Air 5½ percent	—	0.055	—	0.055
Coarse aggregate	$\frac{356}{1.015} = 351$	$\frac{351}{1000 \times 1.34} = 0.262$	356	$\frac{356}{1000 \times 1.35} = 0.264$
Fine aggregate	$\frac{571}{1.04} = 549$	$\frac{549}{1000 \times 1.99} = 0.276$	571	$\frac{571}{1000 \times 1.97} = 0.290$
Added water	$1000 \times 0.275 = 275$	$1.000 - 0.725 = 0.275$	259	$\frac{259}{1000} = 0.259$
Total	1590	1.000	1601	1.000
	Adjusted mix dry basis		Adjusted mix damp basis	
	$m_c = 0$ percent, $S_{c0} = 1.34$	$m_f = 0$ percent, $S_{f0} = 1.99$	$m_c = 5$ percent, $S_{c5} = 1.37$	$m_f = 8$ percent, $S_{f8} = 1.95$
	Weight, kg	Effective displaced volume, m^3	Weight, kg	Effective displaced volume, m^3
Cement	415	0.132	415	0.132
Air 5½ percent	—	0.055	—	0.055
Coarse aggregate	351	0.262	$351 \times 1.05 = 369$	$\frac{369}{1000 \times 1.37} = 0.269$
Fine aggregate	549	0.276	$549 \times 1.08 = 593$	$\frac{593}{1000 \times 1.95} = 0.304$
Added water	275	0.275	$1000 \times 0.240 = 240$	$1.000 - 0.760 = 0.240$
Total	1590	1.000	1617	1.000

tively. Assume relationship between specific gravity factors and moisture contents of the particular coarse and fine aggregates has been predetermined as shown in Fig. 4.2.2. Effective displaced volumes of each component of the mix are calculated as shown, using damp weights of aggregates and the specific gravity factors corresponding to the moisture contents of the aggregates, resulting in a total yield of 27.00 cu ft (1.000 m³).

Assume that in a subsequent shipment of aggregates the moisture contents have changed to $m_c = 5$ percent and $m_f = 8$ percent, and it becomes necessary, therefore, to adjust batch weights of materials to maintain the yield of 27.00 cu ft (1.000 m³). General procedure is to convert the original mix as recorded on a damp aggregate basis to the equivalent mix recorded on the dry aggregate basis, as shown in upper left quadrant of Table 4.3.3a (Table 4.3.3b), and then to adjust from the dry aggregate basis to the new aggregate moisture conditions as shown in lower right quadrant of Table 4.3.3a (Table 4.3.3b).

Detailed procedure to adjust for changes in moisture content of aggregates is as listed below and as shown in Table 4.3.3a (Table 4.3.3b):

(a) Maintain constant the weight of cement and the effective displaced volumes of cement and air

(b) Calculate new weights of both coarse and fine aggregates, using the appropriate value of m, such that the dry weights of both coarse and fine aggregates remain constant

(c) Calculate effective displaced volumes of both coarse and fine aggregates using weights of the aggregates in the appropriate moisture condition and the specific gravity factor corresponding to that moisture condition

(d) Calculate the required effective displaced volume of added water as the difference between

TABLE 4.3.4a—ADJUSTMENT FOR CHANGE IN AGGREGATE PROPORTIONS

	Original mix dry basis		Original mix damp basis	
	m_c = 0 percent, S_{c0} = 1.34	m_f = 0 percent, S_{f0} = 1.99	m_c = 1½ percent, $S_{c1½}$ = 1.35	m_f = 4 percent, S_{f4} = 1.97
	Weight, lb	Effective displaced volume, cu ft	Weight, lb	Effective displaced volume, cu ft
Cement	700	3.56	700	3.56
Air 5½ percent	—	1.49	—	1.49
Coarse aggregate	591	7.07	600	7.12
Fine aggregate	926	7.46	963	7.83
Added water	463	7.42	437	7.00
Total	2680	27.00	2700	27.00
	Adjusted mix dry basis		Adjusted mix damp basis	
	m_c = 0 percent, S_{c0} = 1.34	m_f = 0 percent, S_{f0} = 1.99	m_c = 1½ percent, $S_{c1½}$ = 1.35	m_f = 4 percent, S_{f4} = 1.97
	Weight, lb	Effective displaced volume, cu ft	Weight, lb	Effective displaced volume, cu ft
Cement	$700 - 35 = 665$	$\frac{665}{62.4 \times 3.15} = 3.38$	665	3.38
Air 5½ percent	—	1.49	—	1.49
Coarse aggregate	$8.03 \times 62.4 \times 1.34 = 671$	$0.537 \times 14.95 = 8.03$	$671 \times 1.015 = 681$	$\frac{681}{62.4 \times 1.35} = 8.08$
Fine aggregate	$6.92 \times 62.4 \times 1.99 = 859$	$0.463 \times 14.95 = 6.92$	$859 \times 1.04 = 893$	$\frac{893}{62.4 \times 1.97} = 7.26$
Added water	$463 - 15 = 448$	$\frac{448}{62.4} = 7.18$	$6.79 \times 62.4 = 424$	$27.00 - 20.21 = 6.79$
Total	2643	27.00	2663	27.00

TABLE 4.3.4b—ADJUSTMENT FOR CHANGE IN AGGREGATE PROPORTIONS (METRIC UNITS)

	Original mix dry basis		Original mix damp basis	
	m_c = 0 percent, S_{c0} = 1.34	m_f = 0 percent, S_{f0} = 1.99	m_c = 1½ percent, $S_{c1½}$ = 1.35	m_f = 4 percent, S_{f4} = 1.97
	Weight, kg	Effective displaced volume, m^3	Weight, kg	Effective displaced volume, m^3
Cement	415	0.132	415	0.132
Air 5½ percent	—	0.055	—	0.055
Coarse aggregate	351	0.262	356	0.264
Fine aggregate	549	0.276	571	0.290
Added water	275	0.275	259	0.259
Total	1590	1.000	1601	1.000
	Adjusted mix dry basis		Adjusted mix damp basis	
	m_c = 0 percent, S_{c0} = 1.34	m_f = 0 percent, S_{f0} = 1.99	m_c = 1½ percent, $S_{c1½}$ = 1.35	m_f = 4 percent, S_{f4} = 1.97
	Weight, kg	Effective displaced volume, m^3	Weight, kg	Effective displaced volume, m^3
Cement	$415 - 21 = 394$	$\frac{394}{1000 \times 3.15} = 0.125$	394	0.125
Air 5½ percent	—	0.055	—	0.055
Coarse aggregate	$0.298 \times 1000 \times 1.34 = 399$	$0.537 \times 0.555 = 0.298$	$399 \times 1.015 = 405$	$\frac{405}{1000 \times 1.35} = 0.300$
Fine aggregate	$0.257 \times 1000 \times 1.99 = 511$	$0.463 \times 0.555 = 0.257$	$511 \times 1.04 = 531$	$\frac{531}{1000 \times 1.97} = 0.270$
Added water	$275 - 10 = 265$	$\frac{265}{1000} = 0.265$	$0.250 \times 1000 = 250$	$1.000 - 0.750 = 0.250$
Total	1569	1.000	1580	1.000

the required 27.00 cu ft (1.000 m^3) and the total of the effective displaced volumes of the cement, air, and coarse and fine aggregates

(e) Calculate required weight of added water as 62.4 lb per cu ft (1000 kg/m^3) multiplied by the required effective displaced volume of added water determined in (d).

4.3.4 *Adjustment for change in aggregate proportions*—Assume that an original mix having the batch weights shown in upper part of Table 4.3.4a* (Table 4.3.4b for metric units)* has a satisfactory cement content, air content, slump and yield, but that observations indicate more coarse aggregate may be used without harming workability or finishability.

Total effective displaced volume of aggregates in the original mix is 7.07 + 7.46 = 14.53 cu ft (0.262 + 0.276 = 0.538 m^3), (dry aggregate basis).

Coarse aggregate is 48.7 percent of the total volume of aggregates and fine aggregate is 51.3 percent of the total. Assume that it is desired to increase coarse aggregate by 5 to 53.7 percent of total.

Procedure to adjust for desired change in aggregate proportions is as listed below and as shown in Table 4.3.4a (Table 4.3.4b):

(a) Assume 5 percent decrease in fine aggregate will require 5 × 3 = 15 lb per cu yd (5 × 2 = 10 kg/m^3) decrease in water content (see Section 4.3.2.1). Adjusted water, dry basis, is 463 − 15 = 448 lb (275 − 10 = 265 kg)

(b) Assume 15 lb decrease in water will permit reduction in cement of 0.05 × 700 = 35 lb (0.05 × 415 = 21 kg) (see Section 4.3.2.1). Adjusted cement is 700 − 35 = 665 lb (415 − 21 = 394 kg)

(c) Required effective displaced volume of total aggregates is then 27.00 cu ft (1.000 m^3) minus the sum of the volumes of cement, air, and water = (27.00 − 12.05) = 14.95 cu ft [(1.000 − 0.445) = 0.555 m^3]. Required effective displaced volume of coarse aggregate is then 0.537 × 14.95 = 8.03 cu ft (0.537 × 0.555 = 0.298 m^3), and of fine aggregate 0.463 × 14.95 = 6.92 cu ft (0.463 × 0.555 = 0.257 m^3). These effective displaced volumes are converted to dry weights as shown in lower left quadrant of Table 4.3.4a (Table 4.3.4b)

(d) Convert the adjusted mix on the dry aggregate basis to the adjusted mix on the damp aggregate basis as shown in lower right quadrant of Table 4.3.4a (Table 4.3.4b)

*For illustration the same original mix is assumed as upper part of Table 4.3.3a (Table 4.3.3b). See Section 4.3.3 for details of conversion from damp aggregate basis to dry aggregate basis, and vice versa.

TABLE 4.3.5a—ADJUSTMENT FOR CHANGE IN CEMENT FACTOR

	Original mix dry basis		Original mix damp basis	
	m_c = 0 percent, S_{c0} = 1.34	m_f = 0 percent, S_{f0} = 1.99	m_c = 1½ percent, $S_{c1½}$ = 1.35	m_f = 4 percent, S_{f4} = 1.97
	Weight, lb	Effective displaced volume, cu ft	Weight, lb	Effective displaced volume, cu ft
Cement	700	3.56	700	3.56
Air 5½ percent	—	1.49	—	1.49
Coarse aggregate	591	7.07	600	7.12
Fine aggregate	926	7.46	963	7.83
Added water	463	7.42	437	7.00
Total	2680	27.00	2700	27.00
	Adjusted mix dry basis		Adjusted mix damp basis	
	m_c = 0 percent, S_{c0} = 1.34	m_f = 0 percent, S_{f0} = 1.99	m_c = 1½ percent, $S_{c1½}$ = 1.35	m_f = 4 percent, S_{f4} = 1.97
	Weight, lb	Effective displaced volume, cu ft	Weight, lb	Effective displaced volume, cu ft
Cement	750	$\frac{750}{62.4 \times 3.15} = 3.82$	750	$\frac{750}{62.4 \times 3.15} = 3.82$
Air 5½ percent	—	1.49	—	1.49
Coarse aggregate	591	7.07	600	7.12
Fine aggregate	7.20 × 62.4 × 1.99 = 894	27.00 − 19.80 = 7.20	894 × 1.04 = 930	$\frac{930}{62.4 \times 1.97} = 7.57$
Added water	463	7.42	62.4 × 7.00 = 437	27.00 − 20.00 = 7.00
Total	2698	27.00	2717	27.00

TABLE 4.3.5b—ADJUSTMENT FOR CHANGE IN CEMENT FACTOR (METRIC UNITS)

	Original mix dry basis		Original mix damp basis	
	m_c = 0 percent, S_{c0} = 1.34	m_f = 0 percent, S_{f0} = 1.99	m_c = 1½ percent, $S_{c1\frac{1}{2}}$ = 1.35	m_f = 4 percent, S_{f4} = 1.97
	Weight, kg	Effective displaced volume, m^3	Weight, kg	Effective displaced volume, m^3
Cement	415	0.132	415	0.132
Air 5½ percent	—	0.055	—	0.055
Coarse aggregate	351	0.262	356	0.264
Fine aggregate	549	0.276	571	0.290
Added water	275	0.275	259	0.259
Total	1590	1.000	1601	1.000
	Adjusted mix dry basis		**Adjusted mix damp basis**	
	m_c = 0 percent, S_{c0} = 1.34	m_f = 0 percent, S_{f0} = 1.99	m_c = 1½ percent, $S_{c1\frac{1}{2}}$ = 1.35	m_f = 4 percent, S_{f4} = 1.97
	Weight, kg	Effective displaced volume, m^3	Weight, kg	Effective displaced volume, m^3
Cement	445	$\frac{445}{1000 \times 3.15} = 0.141$	445	0.141
Air 5½ percent	—	0.055	—	0.055
Coarse aggregate	351	0.262	356	0.264
Fine aggregate	$0.267 \times 1000 \times 1.99 = 531$	$1.000 - 0.733 = 0.267$	$531 \times 1.04 = 552$	$\frac{552}{1000 \times 1.97} = 0.280$
Added water	275	0.275	$1000 \times 0.260 = 260$	$1.000 - 0.740 = 0.260$
Total	1602	1.000	1613	1.000

4.3.5 *Adjustment for change in cement factor*—Assume that a mix as shown in the upper part of Table 4.3.5a* (Table 4.3.5b* for metric units) has been used satisfactorily for part of a structure, but that for other parts a desired increase in strength indicates a new cement factor of 750 lb per cu yd (445 kg/m³) to be required.

Procedure to adjust for change in cement factor is as listed below and as shown in Table 4.3.5a (Table 4.3.5b):

(a) Maintain constant the weights of coarse aggregate and added water, and the effective displaced volumes of air, coarse aggregate, and added water

(b) Calculate effective displaced volume for increased weight of cement

(c) Calculate required effective displaced volume of fine aggregate on dry basis as the difference between 27.00 cu ft (1.000 m³) and the sum of the effective displaced volumes of cement, air, coarse aggregate, and added water

(d) Calculate required weight of fine aggregate on the dry basis from the effective displaced volume determined in (c)

(e) Convert the adjusted weight of fine aggregate on the dry aggregate basis to the adjusted weight on the damp aggregate basis as shown in lower right quadrant of Table 4.3.5a (Table 4.3.5b)

(f) Calculate required effective displaced volume of added water and from this calculate the required weight of added water as shown in lower right quadrant of Table 4.3.5a (Table 4.3.5b)

4.3.6 *Adjustment for change in air content*—Assume that an original mix as shown in upper part of Table 4.3.6a* (Table 4.3.6b* for metric units) has 5½ percent air content and that it is desired to increase air content to 7½ percent.

Procedure to adjust for desired change in air content is as listed below and shown in Table 4.3.6a (Table 4.3.6b):

(a) Assume 2 percent increase in air will require $2 \times 5 = 10$ lb per cu yd ($2 \times 3 = 6$ kg/m³) decrease in water content to maintain slump (see Section 4.3.2.2)

*For illustration, the same original mix is assumed as in upper part of Table 4.3.3a (Table 4.3.3b). See Section 4.3.3 for details of conversion from damp aggregate basis to dry aggregate basis and vice versa.

TABLE 4.3.6a—ADJUSTMENT FOR CHANGE IN AIR CONTENT

	Original mix dry basis		Original mix damp basis	
	m_c = 0 percent S_{c0} = 1.34	m_f = 0 percent S_{f0} = 1.99	m_c = 1½ percent $S_{c1½}$ = 1.35	m_f = 4 percent S_{f4} = 1.97
	Weight, lb	Effective displaced volume, cu ft	Weight, lb	Effective displaced volume, cu ft
Cement	700	3.56	700	3.56
Air 5½ percent	—	1.49	—	1.49
Coarse aggregate	591	7.07	600	7.12
Fine aggregate	926	7.46	963	7.83
Added water	463	7.42	437	7.00
Total	2680	27.00	2700	27.00
	Adjusted mix dry basis		Adjusted mix damp basis	
	m_c = 0 percent S_{c0} = 1.34	m_f = 0 percent S_{f0} = 1.99	m_c = 1½ percent $S_{c1½}$ = 1.35	m_f = 4 percent S_{f4} = 1.97
	Weight, lb	Effective displaced volume, cu ft	Weight, lb	Effective displaced volume, cu ft
Cement	700 + 25 = 725	$\frac{725}{62.4 \times 3.15} = 3.69$	725	3.69
Air 7½ percent	—	0.075 × 27 = 2.02	—	2.02
Coarse aggregate	591	7.07	600	7.12
Fine aggregate	6.96 × 62.4 × 1.99 = 864	27.00 − 20.04 = 6.96	864 × 1.04 = 899	$\frac{899}{62.4 \times 1.97} = 7.31$
Added water	463 − 10 = 453	$\frac{453}{62.4} = 7.26$	62.4 × 6.86 = 428	27.00 − 20.14 = 6.86
Total	2633	27.00	2652	27.00

TABLE 4.3.6b—ADJUSTMENT FOR CHANGE IN AIR CONTENT (METRIC UNITS)

	Original mix dry basis		Original mix damp basis	
	m_c = 0 percent S_{c0} = 1.34	m_f = 0 percent S_{f0} = 1.99	m_c = 1½ percent $S_{c1½}$ = 1.35	m_f = 4 percent S_{f4} = 1.97
	Weight, kg	Effective displaced volume, m^3	Weight, kg	Effective displaced volume, m^3
Cement	415	0.132	415	0.132
Air 5½ percent	—	0.055	—	0.055
Coarse aggregate	351	0.262	356	0.264
Fine aggregate	549	0.276	571	0.290
Added water	275	0.275	259	0.259
Total	1590	1.000	1601	1.000
	Adjusted mix dry basis		Adjusted mix damp basis	
	m_c = 0 percent S_{c0} = 1.34	m_f = 0 percent S_{f0} = 1.99	m_c = 1½ percent $S_{c1½}$ = 1.35	m_f = 4 percent S_{f4} = 1.97
	Weight, kg	Effective displaced volume, m^3	Weight, kg	Effective displaced volume, m^3
Cement	415 + 15 = 430	$\frac{430}{1000 \times 3.15} = 0.137$	430	0.137
Air 7½ percent	—	0.075 × 1.000 = 0.075	—	0.075
Coarse aggregate	351	0.262	356	0.264
Fine aggregate	1000 × 0.257 × 1.99 = 511	1.000 − 0.743 = 0.257	511 × 1.04 = 531	$\frac{531}{1000 \times 1.97} = 0.270$
Added water	275 − 6 = 269	$\frac{269}{1000} = 0.269$	1000 × 0.254 = 254	1.000 − 0.746 = 0.254
Total	1561	1.000	1571	1.000

TABLE 4.3.7a—ADJUSTMENT FOR CHANGE IN SLUMP

	Original mix dry basis		Original mix damp basis	
	$m_c = 0$ percent, $S_{c0} = 1.34$	$m_f = 0$ percent, $S_{f0} = 1.99$	$m_c = 1\frac{1}{2}$ percent, $S_{c1\frac{1}{2}} = 1.35$	$m_f = 4$ percent, $S_{f4} = 1.97$
	Weight, lb	Effective displaced volume, cu ft	Weight, lb	Effective displaced volume, cu ft
Cement	700	3.56	700	3.56
Air 5½ percent	—	1.49	—	1.49
Coarse aggregate	591	7.07	600	7.12
Fine aggregate	926	7.46	963	7.83
Added water	463	7.42	437	7.00
Total	2680	27.00	2700	27.00
	Adjusted mix dry basis		Adjusted mix damp basis	
	$m_c = 0$ percent, $S_{c0} = 1.34$	$m_f = 0$ percent, $S_{f0} = 1.99$	$m_c = 1\frac{1}{2}$ percent, $S_{c1\frac{1}{2}} = 1.35$	$m_f = 4$ percent, $S_{f4} = 1.97$
	Weight, lb	Effective displaced volume, cu ft	Weight, lb	Effective displaced volume, cu ft
Cement	$700 + 42 = 742$	$\frac{742}{62.4 \times 3.15} = 3.77$	742	3.77
Air 5½ percent	—	1.49	—	1.49
Coarse aggregate	591	7.07	600	7.12
Fine aggregate	$6.93 \times 62.4 \times 1.99 = 861$	$27.00 - 20.07 = 6.93$	$861 \times 1.04 = 895$	$\frac{895}{62.4 \times 1.97} = 7.28$
Added water	$463 + 20 = 483$	$\frac{483}{62.4} = 7.74$	$62.4 \times 7.34 = 458$	$27.00 - 19.66 = 7.34$
Total	2677	27.00	2695	27.00

TABLE 4.3.7b—ADJUSTMENT FOR CHANGE IN SLUMP (METRIC UNITS)

	Original mix dry basis		Original mix damp basis	
	$m_c = 0$ percent, $S_{c0} = 1.34$	$m_f = 0$ percent, $S_{f0} = 1.99$	$m_c = 1\frac{1}{2}$ percent, $S_{c1\frac{1}{2}} = 1.35$	$m_f = 4$ percent, $S_{f4} = 1.97$
	Weight, kg	Effective displaced volume, m^3	Weight, kg	Effective displaced volume, m^3
Cement	415	0.132	415	0.132
Air 5½ percent	—	0.055	—	0.055
Coarse aggregate	351	0.262	356	0.264
Fine aggregate	549	0.276	571	0.290
Added water	275	0.275	259	0.259
Total	1590	1.000	1601	1.000
	Adjusted mix dry basis		Adjusted mix damp basis	
	$m_c = 0$ percent, $S_{c0} = 1.34$	$m_f = 0$ percent, $S_{f0} = 1.99$	$m_c = 1\frac{1}{2}$ percent, $S_{c1\frac{1}{2}} = 1.35$	$m_f = 4$ percent, $S_{f4} = 1.97$
	Weight, kg	Effective displaced volume, m^3	Weight, kg	Effective displaced volume, m^3
Cement	$415 + 25 = 440$	$\frac{440}{1000 \times 3.15} = 0.140$	427	0.136
Air 5½ percent	—	0.055	—	0.055
Coarse aggregate	351	0.262	356	0.264
Fine aggregate	$1000 \times 0.256 \times 1.99 = 509$	$1.000 - 0.744 = 0.256$	$509 \times 1.04 = 529$	$\frac{529}{1000 \times 1.97} = 0.269$
Added water	$275 + 12 = 287$	$\frac{287}{1000} = 0.287$	$1000 \times 0.276 = 276$	$1.000 - 0.724 = 0.276$
Total	1587	1.000	1588	1.000

(b) Assume increase in air content will require an additional 25 lb of cement per cu yd (15 kg/m^3) to maintain strength (see Section 2.4.2)

(c) Adjust effective displaced volume of fine aggregate to maintain yield at 27.00 cu ft (1.000 m^3) as shown in lower left quadrant of Table 4.3.6a (Table 4.3.6b)

(d) Convert adjusted mix on dry basis to adjusted mix on damp basis as shown in lower right quadrant of Table 4.3.6a (Table 4.3.6b)

4.3.7 *Adjustment for change in slump*—Assume that an original mix as shown in upper part of Table 4.3.7a* (Table 4.3.7b* for metric units) has a slump of 3 in. (75 mm) and that it is desired to increase slump to 5 in. (125 mm) without changing compressive strength.

Procedure to adjust for desired change in slump is as listed below and shown in Table 4.3.7a (Table 4.3.7b):

(a) Assume 2 in. (50 mm) increase in slump will require $2 \times 10 = 20$ lb increase in water content per cu yd ($50/25 \times 6 = 12$ kg/m^3) (see Section 4.3.2.3)

(b) Assume cement factor must be increased by $2 \times 0.03 \times 700 = 42$ lb per cu yd ($2 \times 0.03 \times 415 = 25$ kg/m^3) to maintain strength (see Section 4.3.2)

(c) Adjust effective displaced volume of fine aggregate to maintain yield at 27.00 cu ft (1.000 m^3) as shown in lower left quadrant of Table 4.3.7a (Table 4.3.7b)

(d) Convert adjusted mix on dry basis to adjusted mix on damp basis as shown in lower right quadrant of Table 4.3.7a (Table 4.3.7b)

4.4—Controlling proportions in the field

Proportions which have been established for given conditions may require adjustment from time to time to maintain the planned proportions in the field. Knowledge that proportions are remaining essentially constant, or that they may be varying beyond acceptable limits, can be obtained by conducting tests for fresh unit weight of concrete, air content, and slump. These tests should be made not only at such uniform frequency as may be specified (a given number of tests per stated quantity of concrete, per stated time period or per stated section of structure, etc.), but should also be made when observation indicates some change in the ingredients of the concrete or in the physical characteristics of the concrete. These tests should be made, for example, when moisture content of the aggregates may have changed appreciably, when the concrete shows change in slump or workability characteristics, or when there is an appreciable change in added water requirement.

A change in fresh unit weight of concrete, with batch weights and air content remaining constant, shows that the batch is over yielding (with lower unit weight) or under yielding (with higher unit weight). The over yielding batch will have lower than planned cement content and the under yielding batch will have higher than planned cement content.

A change in fresh unit weight of concrete indicates (a) a batching error, (b) a change in air content, (c) a change in the specific gravity factor of one or both of the coarse or fine aggregates. A change in specific gravity factor may be due to (a) a change in the aggregate moisture content, or (b) a basic change in the aggregate density. Air tests, aggregate moisture tests, and specific gravity factor determinations will establish the cause of such changes (see also Fig. 4.4).

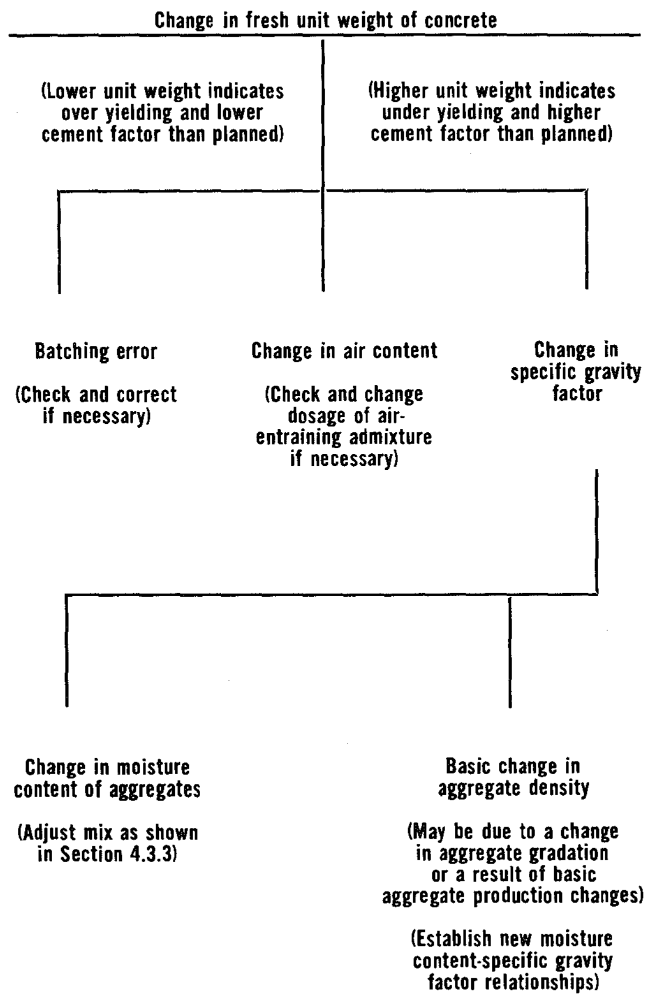

Fig. 4.4—Controlling proportions

*For illustration the same original mix is assumed as upper part of Table 4.3.3a (Table 4.3.3b). See Section 4.3.3 for details of conversion from damp aggregate basis to dry aggregate basis and vice versa.

A change in aggregate specific gravity factor may be the result of (a) a change in the moisture content of the aggregate, or (b) a basic change in aggregate density. If a moisture test indicates moisture changes, the mix should be adjusted as shown in Section 4.3.3. If the basic aggregate density has changed, determination of new moisture content — specific gravity factor relationships are indicated. (Aggregate density changes may be a result of changes in raw material and/or its processing.) A change in slump may indicate (a) a change in air content, (b) a change in moisture content of aggregate without corresponding change in batching, or (c) a change in aggregate gradation or density. Each of these factors is also indicated by the fresh unit weight test.

Note: Control of concrete mixes in the field also requires recognition of possible changes due to changes in ambient temperature, changes in temperature of ingredients, length of mixing and agitating time, and other causes. Discussion of such factors is beyond the scope of this standard.

APPENDIX A—DETERMINATION OF SPECIFIC GRAVITY FACTORS OF STRUCTURAL LIGHTWEIGHT AGGREGATE[1]

Methods presented here describe procedures for determining the specific gravity factors of lightweight aggregates, either dry or moist.

PYCNOMETER METHOD FOR FINE AND COARSE LIGHTWEIGHT AGGREGATES

Apparatus

(a) A pycnometer consisting of a narrow mouth 2-qt Mason jar with a spun-brass pycnometer top (Soiltest G-335, Humboldt H-3380, or equivalent).

(b) A balance or scale having a capacity of at least 5 kg and a sensitivity of 1 g.

(c) A water storage jar of about 5 gal. capacity, for maintaining water at room temperature.

(d) Isopropyl (rubbing) alcohol and a medicine dropper.

Calibration of the pycnometer

The pycnometer is filled with water and agitated to remove any entrapped air. The filled pycnometer is then weighed and the weight (weight B in grams) is recorded.

Sampling procedure

Representative samples of about 2 to 3 cu ft of each size of aggregate should be obtained from the stockpile and put through a sample splitter or quartered until the correct size of sample desired has been obtained. During this operation with damp aggregates, extreme care is necessary to prevent the aggregates from drying. The aggregate sample should occupy ½ to ⅔ of the volume of the 2-qt pycnometer.

Test procedure

Two representative samples should be obtained of each size of lightweight aggregate to be tested.

The first is weighed, placed in an oven at 105 to 110 C (221 to 230 F), and dried to constant weight. "Frying pan drying" to constant weight is an acceptable field expedient. The dry aggregate weight is recorded and the aggregate moisture content (percent of aggregate dry weight, 100 m) is calculated.

The second aggregate sample is weighed (weight C in grams). The sample is then placed in the empty pycnometer and water is added until the jar is ¾ full. The time of water addition should be noted.

The air entrapped between the aggregate particles is removed by rolling and shaking the jar. During agitation, the hole in the pycnometer top is covered with the operator's finger. The jar is then filled and again agitated to eliminate any additional entrapped air. If foam appears during the agitation and prevents the complete filling of the pycnometer with water at this stage, a *minimum* amount of the isopropyl alcohol should be added with the medicine dropper to eliminate the foam. The water level in the pycnometer must be adjusted to full capacity and the exterior surfaces of the jar must be dried before weighing.

The pycnometer, thus filled with sample and water, is weighed (weight A in grams) afer 5, 10, and 30 min of sample immersion to obtain complete data, and the weights at these times are recorded.

Calculation

The pycnometer specific gravity factor, S, after any particular immersion time, is calculated by the following formula:

$$S = \frac{C}{C + B - A}$$

where

A = weight of pycnometer charged with aggregate and then filled with water, g

B = weight of pycnometer filled with water, g

C = weight of aggregate tested, moist or dry, g

BUOYANCY METHODS FOR COARSE AGGREGATES

If larger test samples of coarse aggregate than can be evaluated in the pycnometer are desired, coarse aggregate specific gravity factors may be determined by the wholly equivalent weight-in-air-and-water procedures described in ASTM C 127. The top of the container used for weighing the aggregates under water must be closed with a screen to prevent light particles from floating away from the sample.

Specific gravity factors by this method are calculated by the equation:

$$\text{Specific gravity factor, } S = \frac{C}{C - E}$$

where

C = same as above (the weight in air)
E = weight of coarse aggregate sample under water, g
S = specific gravity factor, equal (by the theory of the method) to the pycnometer specific gravity factor

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This report was submitted to letter ballot of the committee which consists of 36 members; 25 members returned their ballots, all of whom voted affirmatively.

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ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction, and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be incorporated directly into the Project Documents.

ACI STANDARD

Recommended Practice for Selecting Proportions for No-Slump Concrete (ACI 211.3-75)*

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This standard is intended as a supplement to ACI Standard "Recommended Practice for Selecting Proportions for Normal and Heavyweight Concrete (ACI 211.1-74)." The standard describes a procedure for proportioning concretes having slumps in the range of zero to 1 in. and consistencies below this range, for aggregates up to 1½ in. maximum size. Suitable equipment for measuring such consistencies is described. Tables similar to those in ACI 211.1-74 are provided which, along with laboratory tests on physical properties of fine and coarse aggregate, yield information for obtaining concrete proportions for a trial mixture. Examples of the use of these tables, in conjunction with tables in ACI 211.1-74 are given. An appendix provides complete conversion of all tabular data to metric system units and shows a sample problem worked out with metric values.

Keywords: aggregate size; aggregates; coarse aggregates; compaction tests; concrete durability; concretes; **consistency tests;** fine aggregates; measuring instruments; **mix proportioning; no-slump concrete; slump tests; test equipment;** water-cement ratio.

*Adopted as a standard of the American Concrete Institute Jan. 1975, to supersede ACI 211-65, in accordance with the Institute's standardization procedure.
†Members of Subcommittee 2 which prepared this report.

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CONTENTS

CHAPTER 1—SCOPE AND LIMITS

The ACI standard "Recommended Practice for Selecting Proportions for Normal Weight and Heavyweight Concrete (ACI 211.1-74)"[1] presents the details of a method for proportioning concrete having consistencies, as measured by the slump test, in the range of 1 to 7 in. The present standard is an extension of ACI 211.1-74 which will facilitate proportioning concretes having drier consistencies (slump measurements of 1 in. or less). Three possible methods of measuring these consistencies are described, since the slump test is impractical for these drier consistencies.

CHAPTER 2—PRELIMINARY CONSIDERATIONS

2.1—General

2.1.1—The general comments contained in the introduction to ACI 211.1-74 are pertinent to the procedures discussed in this standard. The description of the make-up of concrete, the possible differences in properties of the ingredients from different sources, and the need for a knowledge of some of the physical properties of the aggregates and the cement apply equally to this standard.

2.2—Methods for measuring consistency

2.2.1—Workability is that property of concrete which determines the ease with which it can be mixed, placed, consolidated, and finished. There is no one test known at this time which will measure this property in quantitative terms. It is usually expedient to use some type of consistency measurement as an index to workability. Consistency may be defined as the ability of freshly mixed concrete to flow. The slump test is the most familiar one in the United States and is the basis for the measures of consistencies shown in ACI 211.1-74.

2.2.2 — Translating a particular consistency measurement into a determination of whether workability is adequate is based on judgment and

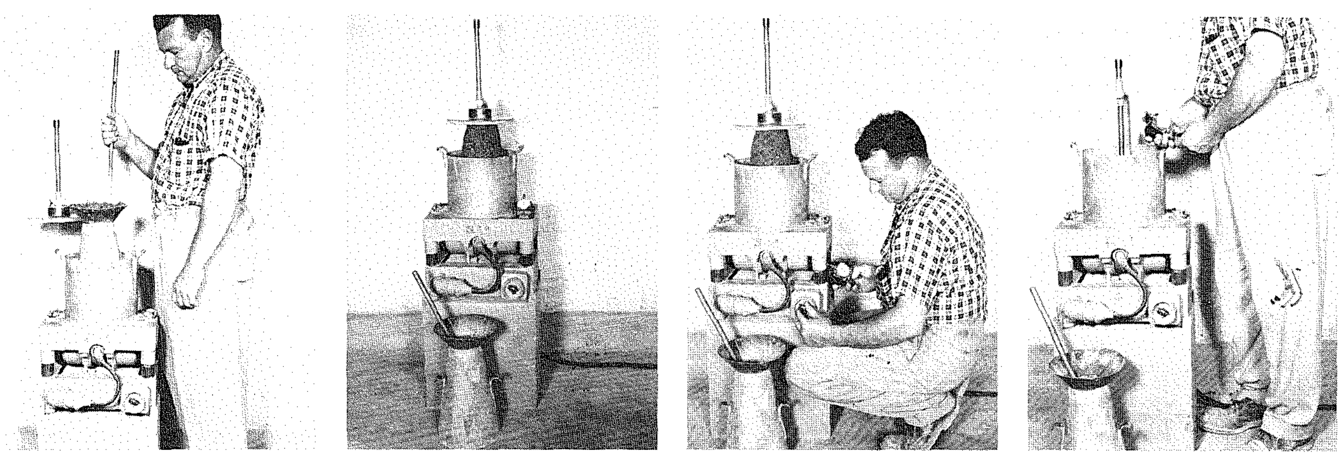

Fig. 2.2.3—Vebe apparatus

depends on the method of compaction being used. No-slump concrete will have poor workability if compaction by hand rodding is attempted. If vibration is used, however, such a concrete might be considered as having excellent workability characteristics. The range of workable mixtures can therefore be widened by adopting compaction techniques which impart greater energy into the mass to be consolidated. The Vebe apparatus,[2,3] the compacting factor apparatus,[4,8] and the drop table developed by Thaulow,[5] described in the following paragraphs, are devices which can provide a useful measure of the consistency for concrete mixtures with less than 1-in. slump. A more detailed description of each is presented in the appendix. At this time, no such methods have been standardized for use in the United States. Of the three, the Vebe apparatus appears to be the most suitable for obtaining a measure of consistency for the concretes described in this standard. If none of these methods is available, compaction of the trial mixture under actual placing conditions in the field or laboratory will, of necessity, serve as a means for determining whether the consistency and workability are adequate. In this case, however, it is likely that the first selected job mixture will not be as close to the final mixture as one selected when using either of the three suggested methods of measuring consistency.

2.2.3 *Vebe apparatus*—The operation of the Vebe apparatus is shown in Fig. 2.2.3. The main parts are a vibrating table, a sample container, slump cone, and a plastic plate and graduated rod serving as a surcharge weight and end-point reference. The measure of consistency is the time of vibration in seconds required to change the shape of the truncated cone of concrete, left standing after removal of the slump cone, into that of a cylinder with a level top surface. This time is presumed directly proportional to the energy used to compact the sample. In very dry mixtures, this method appears to be the most sensitive in determining differences in consistency.

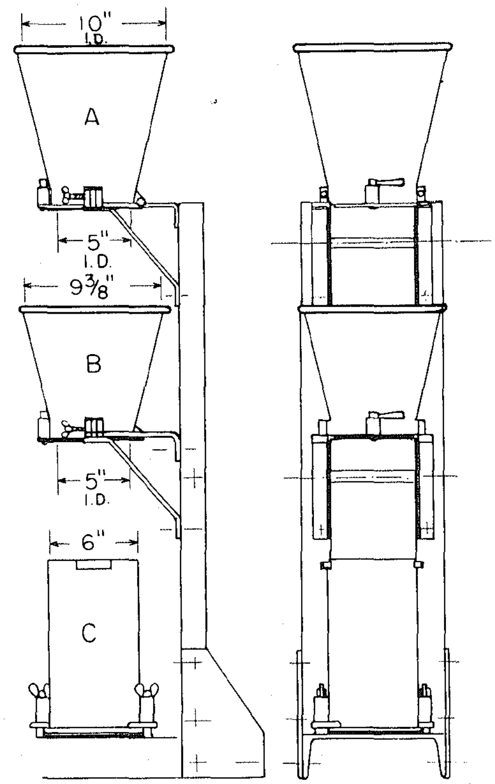

Fig. 2.2.4—Compacting factor apparatus

2.2.4 *Compacting factor*—This method is described in British Standard 1881.[8] The details of the compacting factor apparatus are shown in Fig. 2.2.4. The upper hopper is carefully filled with a sample of the freshly-mixed concrete using a scoop. The sample is dropped through a trap door into the somewhat smaller hopper below, and then dropped by gravity into the 6 x 12-in. cylinder mold below. After strike-off of the cylinder mold, the weight of the concrete in the mold

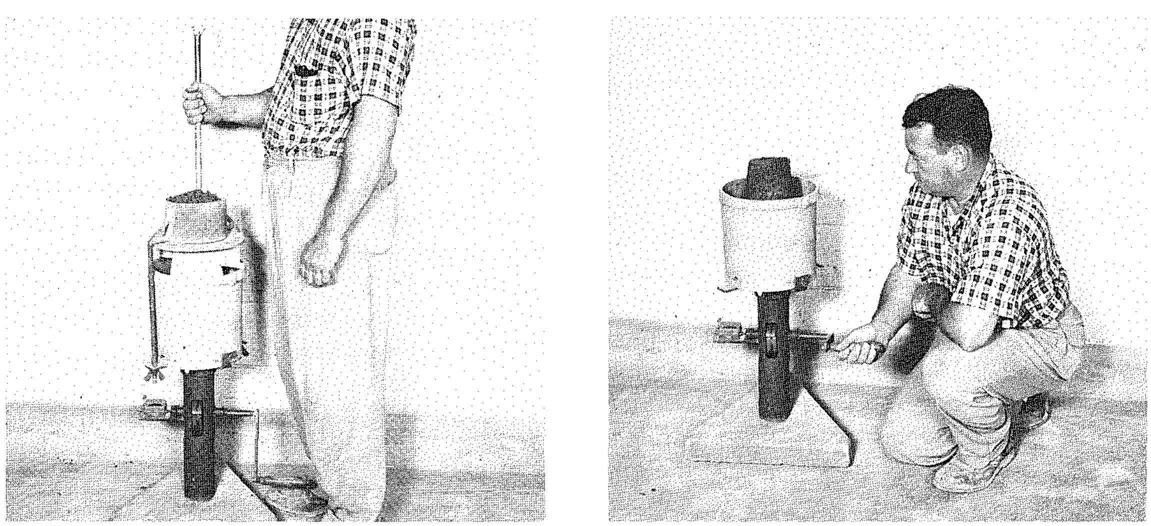

Fig. 2.2.5—Thaulow drop table

TABLE 2.3.1(a)—COMPARISON OF CONSISTENCY MEASUREMENTS BY VARIOUS METHODS

Consistency description	Slump, in.	Vebe, sec	Compacting factor, average	Thaulow drop table, revolutions
Extremely dry	—	32 to 18	—	112 to 56
Very stiff	—	18 to 10	0.70	56 to 28
Stiff	0 to 1	10 to 5	0.75	28 to 14
Stiff plastic	1 to 3	5 to 3	0.85	14 to 7
Plastic	3 to 5	3 to 0	0.90	<7
Flowing	5 to 7	—	0.95	—

is determined. The ratio of this weight to the weight of concrete from the same batch fully compacted by heavy vibration in the mold is called the compacting factor. The test is sensitive for mixtures having a very stiff consistency (Table 2.3.1a) and for wetter mixtures. It is not as sensitive as the Vebe method for extremely dry consistencies.

2.2.5 *Thaulow drop table*—The operation of the drop table is illustrated in Fig. 2.2.5. Like the Vebe, the test consists of transforming a truncated cone mold by rodding and subsequent drops of the table prior to strike-off. The energy for transformation is obtained by successive drops of the table and the measure used to characterize the consistency is the number of revolutions of the hand crank (4 drops per revolution). While not as sensitive as the Vebe in the very dry consistencies, this simple apparatus appears to have merit.

2.2.6—The interrelationship of these methods is shown in Table 2.3.1(a). Note that the Vebe or the drop table can provide a measure of consistency in mixtures termed extremely dry.

2.3—Mixing water requirement

2.3.1—In ACI 211.1-74, Table 5.3.3, approximate mixing water requirements are given for concretes conforming to the consistency descriptions of stiff plastic, plastic, and flowing, as shown in Table 2.3.1(a) of this standard. Considering the water requirement for the 3 to 4-in. slump as 100 percent, the relative water contents for these three consistencies are 92 percent, 100 percent, and 106 percent, respectively. Thaulow[6] extended this concept of average relative water contents to include stiffer mixtures. These are shown in Table 2.3.1(b), establishing the 3 to 4-in. slump classification as 100 percent, and comparing where possible with the ACI 211.1-74 values. Based on this relationship between approximate relative water content and the six consistency designations, Table 2.3.1(c) has been prepared showing the approximate mixing water requirements in pounds per cubic yard using the relative water contents shown by Thaulow for the stiff, very stiff, and extremely dry consistencies. The table also shows the present mixing water requirements in Table 5.3.3 of ACI 211.1-74 for convenience.

TABLE 2.3.1(b)—APPROXIMATE RELATIVE WATER CONTENT FOR DIFFERENT CONSISTENCIES

Consistency description	Approximate relative water content, percent	
	Thaulow[6]	Table 5.3.3 ACI 211.1-74
Extremely dry	78	—
Very stiff	83	—
Stiff	88	—
Stiff plastic	93	92
Plastic	100	100
Flowing	108	106

2.3.2—In a series of laboratory tests conducted for the committee, limited to maximum sizes of aggregate of ⅜ in., ¾ in., and 1½ in., the relative water contents for the six different levels of consistencies compared favorably with those suggested by Thaulow. Consistency measurements included the slump test, the Vebe test, and the drop table. The committee feels that this confirmation, admittedly limited, is sufficient to justify the recommendations shown in Table 2.3.1(c). These are recommended *approximate* mixing water requirements. It must be remembered that for a given combination of materials, a number of factors will contribute to the actual mixing water requirements and may result in a considerable difference from the value shown in Table 2.3.1(c). These will include the particle shape and grading of the aggregate, temperature of the concrete, the effectiveness of mixing, and the method of compaction. With respect to mixing, for example, spiral blade batch and pan-type mixers are more effective for no-slump concretes than are rotating drum mixers.

CHAPTER 3—SELECTING PROPORTIONS

3.1—General

3.1.1—As recommended in ACI 211.1-74, concrete should be placed using the minimum quantity of mixing water consistent with mixing, placing,

TABLE 2.3.1(c)—APPROXIMATE MIXING WATER REQUIREMENTS FOR DIFFERENT CONSISTENCIES AND MAXIMUM SIZES OF AGGREGATES*

Consistency					Relative water content, percent	Water, lb per cu yd for indicated maximum sizes of coarse aggregate				
Description	Slump, in.	Vebe, sec	Drop table, revolutions	Compacting factor		⅜ in.	½ in.	¾ in.	1 in.	1½ in.
Non-air-entrained concrete										
Extremely dry	—	32-18	112-56	—	78	300	285	265	250	235
Very stiff	—	18-10	56-28	0.70	83	315	310	285	265	250
Stiff	0-1	10- 5	28-14	0.75	88	335	325	300	285	265
Stiff plastic	1-3	5- 3	14- 7	0.85	92	350	335	315	300	275
Plastic	3-5	3- 0	<7	0.91	100	385	365	340	325	300
Flowing	5-7	—	—	0.95	106	410	385	360	340	315
Approximate amount of entrapped air in non-air-entrained concrete, percent						3	2.5	2	1.5	1
Air-entrained concrete										
Extremely dry	—	32-18	112-56	—	78	265	250	235	225	210
Very stiff	—	18-10	56-28	0.70	83	285	265	250	235	225
Stiff	0-1	10- 5	28-14	0.75	88	300	285	265	250	235
Stiff plastic	1-3	5- 3	14- 7	0.85	92	305	295	280	270	250
Plastic	3-5	3- 0	<7	0.91	100	340	325	305	295	275
Flowing	5-7	—	—	0.95	106	365	345	325	310	290
Recommended average total air content, percent†						8	7	6	5	4.5

*These quantities of mixing water are for use in computing cement factors for trial batches. They are for reasonably well-shaped angular coarse aggregates graded within limits of accepted specifications.

If *more* water is required than shown, the cement factor, estimated from these quantities, should be increased to maintain desired water-cement ratio, except as otherwise indicated by laboratory tests for strength.

If *less* water is required than shown, the cement factor, estimated from these quantities, *should not* be decreased except as indicated by laboratory tests for strength.

†For consistencies below 1 in. slump, the volume of air entrained by either an air-entraining cement or the usual amount of air-entraining admixture used for more plastic mixtures may be significantly lower than those shown. For these mixtures, it is recommended that the air content resulting from the use of air-entraining cement or the usual amount of air-entraining admixture per unit of cement for more plastic mixtures be accepted as adequate for insuring durability. In the absence of such information for a particular air-entraining admixture, the amount to use per unit of cement can be determined on a trial mix having a slump in the 3 to 4 in. range, or by determining the amount needed to obtain 19 ± 3 percent air in mortar prepared in accordance with ASTM C 185.[7]

and consolidating and finishing requirements, since this will have a favorable influence on strength, durability, and other physical properties. The major considerations in selecting proportions apply equally well to no-slump concretes as to the more plastic mixtures. These considerations are: (1) provide adequate durability to withstand satisfactorily the weather and other destructive agencies to which it may be exposed; (2) produce the strength required to withstand the loads to be imposed without danger of failure; (3) use of the maximum size of aggregate consistent with economic availability, satisfactory placement, and concrete strength; and (4) use of the stiffest consistency which can be compacted efficiently to a homogeneous mass.

3.2—Slump and maximum size of aggregate

3.2.1—Table 5.3.1 of ACI 211.1-74 contains recommendations for consistencies in the range of "stiff plastic" to "flowing." These as well as stiffer consistencies are included in Table 2.3.1 (c). Consistencies in the stiff range or lower are often used in the fabrication of concrete pipe and various precast elements, both prestressed and conventionally reinforced. However, there doesn't appear to be any justification for setting fixed limits for maximum and minimum consistency in these types of construction. The optimum consistency for such work is closely dependent on the methods and equipment in use at a particular plant, and the diversity of available facilities is such that a recommended range for one plant operation may not be suitable for another. The committee does recommend, however, that wherever possible the consistencies used should be in the stiff range or lower, since the use of these drier consistencies, adequately compacted, will result in an improved and more economical product.

3.2.2—The maximum size of aggregate to be selected for a particular type of construction is dictated primarily by consideration of both the minimum dimension of a section and the minimum clear spacing between reinforcing bars, prestressing tendons, ducts for post-tensioning tendons, or other embedded items. The largest permissible maximum size of aggregate should be used, unless smaller sizes are available and their use would result in equal or greater strength at no detriment to other concrete properties, or if smaller sizes would result in improved durability of the concrete.

TABLE 3.4.1—MAXIMUM PERMISSIBLE WATER-CEMENT RATIOS FOR CONCRETE IN SEVERE EXPOSURES*

Type of structure	Structure wet continuously or frequently and exposed to freezing and thawing†	Structure exposed to seawater or sulfates
Thin sections (railings, curbs, sills, ledges, ornamental work) and sections with less than 1 in. cover over steel	0.45	0.40‡
All other structures	0.50	0.45‡

*Based on report of ACI Committee 201, "Durability of Concrete in Service," ACI Journal, *Proceedings* V. 59, No. 12, Dec. 1962, pp. 1771-1820.
†Concrete should also be air-entrained.
‡If sulfate resisting cement (Type II or Type V of ASTM C 150) is used, permissible water-cement ratio may be increased by 0.05.

3.3—Estimating water requirements

3.3.1—The quantity of water per unit volume of concrete required to produce a mixture of the desired consistency is influenced by the maximum size, particle shape and grading of the aggregate, and by the amount of entrained air. It is relatively unaffected by the quantity of cement below about 600 to 650 lb per cu yd. In mixtures richer than these, mixing water requirements may increase significantly as cement content is increased. Guides to acceptable aggregate gradings are available in the recommendations of such organizations as the American Society for Testing and Materials, the American Association of State Highway Officials, in Federal Specifications, and in the requirements of local bodies such as state highway departments, counties, and cities.

3.3.2—The quantities of water shown in Table 2.3.1 (c) of this standard are sufficiently accurate for preliminary estimates of proportions. If, with a particular combination of materials, the water requirement is higher than indicated, the cement content should be increased to maintain the desired water-cement ratio, unless otherwise indicated by laboratory tests. Examples of such adjustment are given later and can also be found in ACI 211.1-74.

3.3.3—Some materials may require less water than indicated in Table 2.3.1 (c). Unless supported by laboratory tests for strength, no adjustment should be made in cement content, since other compensating factors may be involved. For example, a rounded gravel and a normally angular coarse aggregate, both well and similarly graded and of good quality, usually will produce concrete of about the same compressive strength for the same cement factor in spite of differences in water-cement ratio. Also, for the same proportions, different cements may produce concretes having strengths which differ.

3.4—Selecting water-cement ratio

3.4.1—Table 3.4.1, reprinted from ACI 211.1-74, shows the maximum permissible water-cement ratios for different types of structures and degrees of exposure. These values, based on an extensive background of actual field performance and laboratory studies, and intentional air entrainment will provide the proper quality of cement pastes. In addition, other factors such as mixing, placing, consolidating, finishing, curing, quality of ingredients, etc., influence durability and must be suitably controlled to insure that the concrete will be durable.

3.4.2—Intentionally entrained air, as provided by acceptable air-entraining admixtures or air-entraining additions interground to make air-entraining cements, is of great benefit in insuring durable concrete, in addition to providing other advantages, and should always be used when exposure to weathering is expected to be severe. Note in Table 3.4.1 that for all concretes exposed to a severe range in temperature, or frequent alternations of freezing and thawing, intentionally entrained air should be used.

3.4.3—In addition to durability, the selection of water-cement ratio is dependent on the strength required. This can best be determined by laboratory tests made with the same materials, including cement, as will be used in the work. However, if it is not practicable to make such detailed tests, Tables 2.3.1(c) and 3.4.3 afford a basis for estimating water and cement requirements. Table 3.4.3 reproduces the information in Table 5.3.4(a) of ACI 211.1-74 and provides additional information for lower water-cement ratios. The strengths shown are conservative average strengths for the various water-cement ratios.

3.4.4—Using the maximum permissible water-cement ratio from Table 3.4.1 or Table 3.4.3 and the approximate mixing water requirement from Table 2.3.1(c), the cement factor required can be calculated by dividing the pounds of mixing water required per cubic yard by the water-cement ratio, by weight. If the specifications for the job at hand contain a minimum cement factor requirement, the corresponding water-cement ratio for estimating strength can be computed by dividing the pounds of water per cubic yard by the cement factor in pounds per cubic yard. The lowest of the three water-cement ratios—those for durability, strength, or cement factor—should be selected for calculating concrete proportions.

3.4.5—In Table 3.4.3 at equal water-cement ratios, the strengths for the air-entrained concretes are about 20 percent lower than for the non-air-entrained concretes. These differences may not be as great in the no-slump mixtures, since the volume of entrained air in these mixtures using an air-entraining cement or the usual amount of air-entraining admixture per unit of cement will be reduced significantly with no sacrifice in durability.[7] In addition, when cement content and consistency are maintained constant, the differences in strength are partially or entirely offset by reduction of mixing water requirements which result from air entrainment.

3.4.6—The required average strength necessary to ensure the design strength specified for a particular job will be dependent on the degree of control over all operations involved in the production and testing of the concrete. For a complete guide in this respect, see ACI 214-65 "Recommended Practice for Evaluation of Compression Test Results of Field Concrete." If flexural strength is a requirement, rather than compressive strength, the relationship between water-cement ratio and flexural strength should be determined by laboratory tests using the job materials.

TABLE 3.4.3—RELATIONSHIPS BETWEEN WATER-CEMENT RATIO AND COMPRESSIVE STRENGTH OF CONCRETE

Compressive strength at 28 days, psi*	Water-cement ratio, by weight: Non-air-entrained concrete	Air-entrained concrete
7000	0.33	—
6000	0.41	0.32
5000	0.48	0.40
4000	0.57	0.48
3000	0.68	0.59
2000	0.82	0.74

*Values are estimated average strengths for concrete containing not more than the percentage of air shown in Table 2.3.1(c). For a constant water-cement ratio, the strength of concrete is reduced as the air content is increased. See also Section 3.4.5.
Strength is based on 6 x 12 in. cylinders moist-cured 28 days at 73.4 ± 3 F (23 ± 1.7 C) in accordance with Section 9(b) of ASTM C 31 for Making and Curing Concrete Compression and Flexure Test Specimens in the Field.
Relationship assumes maximum size of aggregate about ¾ to 1 in.; for a given source, strength produced for a given water-cement ratio will increase as maximum size of aggregate decreases.

3.5—Estimate of quantity of coarse aggregate

3.5.1—The largest quantity of coarse aggregate per unit volume of concrete should be used, con-

TABLE 3.5.1(a)—VOLUME OF COARSE AGGREGATE PER UNIT OF VOLUME OF CONCRETE OF PLASTIC CONSISTENCY (3-5 IN. SLUMP)

Maximum size of aggregate, in.	Volume of dry-rodded coarse aggregate* per unit volume of concrete for different fineness moduli of sand: 2.40	2.60	2.80	3.00
⅜	0.50	0.48	0.46	0.44
½	0.59	0.57	0.55	0.53
¾	0.66	0.64	0.62	0.60
1	0.71	0.69	0.67	0.65
1½	0.75	0.73	0.71	0.69

*Volumes are based on aggregates in dry-rodded condition as described in ASTM C 29 for Unit Weight of Aggregate.
These volumes are selected from empirical relationships to produce concrete with a degree of workability suitable for usual reinforced construction.

sistent with adequate placeability and workability. For a given aggregate, the amount of mixing water required will then be at a minimum and strength at a maximum. This quantity of coarse aggregate can best be determined from laboratory investigations, using the materials for the intended work, with later adjustment in the field or plant. If such data are not available or cannot be obtained, Table 3.5.1(a) (from Table 5.3.6, ACI 211.1-74) provides a good estimate of the amount of coarse aggregate for various concretes having a degree of workability suitable for usual reinforced construction (approximately 3 to 4-in. slump). These values of dry-rodded volume of coarse aggregate per unit volume of concrete are based on established empirical relationships for aggregates graded within conventional limits. Changes in the consistency of the concrete can be effected by changing the amount of coarse aggregate per unit volume of concrete. As greater amounts of coarse aggregate per unit volume are used, the consistency will be decreased. For the "flowing" and "plastic" consistencies, the volume of coarse aggregate per unit volume of concrete are essentially unchanged from those shown in Table 3.5.1(a). For the stiffer consistencies, those requiring vibration for compaction, the amount of coarse aggregate that can be accommodated increases rather sharply in relation to the amount of fine aggregate required. Table 3.5.1(b) shows some typical values of the volume of coarse aggregate per unit volume of concrete for different consistencies expressed as a percentage of the values shown in Table 3.5.1(a). The information contained in these two tables provides a basis for selecting an appropriate amount of coarse aggregate for the first trial mixture. Adjustments in this amount will probably be necessary in the field or plant operation.

3.5.2—Concrete of comparable workability can be expected with aggregates of comparable size, shape, and grading when a given dry-rodded volume of coarse aggregate per unit volume of concrete is used. In this case of different types of aggregates, particularly those with different particle shapes, the use of a fixed dry-rodded volume of coarse aggregate automatically makes allowance for differences in mortar requirements as reflected by void content of coarse aggregate. For example, angular aggregates have a higher void content; therefore require more mortar than rounded aggregates. The procedure does not reflect variations in grading of coarse aggregates within different maximum size limits, except as they are reflected in percentage of voids. However, for coarse aggregates falling within the limits of conventional grading specifications, this omission is probably of little practical importance. It will be seen that the optimum dry-rodded volume of coarse aggregate per unit volume of concrete depends on its maximum size and the fineness modulus of the fine aggregate as indicated in Table 3.5.1(a).

CHAPTER 4—COMPUTATION OF PROPORTIONS

4.1—General design criteria

4.1.1—Computation of proportions will be explained by one example. The following design criteria are assumed.

4.1.1.1 Non-air-entraining cement will be used and its specific gravity is assumed to be 3.15.

4.1.1.2 Coarse and fine aggregates in each case are of satisfactory quality and are graded within limits of generally accepted specifications.

4.1.1.3 The coarse aggregate has a specific gravity, bulk dry, of 2.68 and an absorption of 0.5 percent.

TABLE 3.5.1(b)—VOLUME OF COARSE AGGREGATE PER UNIT VOLUME OF CONCRETE FOR DIFFERENT CONSISTENCIES*

Consistency					Volume of dry-rodded coarse aggregate per unit volume of concrete for maximum size of aggregate shown [expressed as a percentage of the values shown in Table 3.5.1(a)].				
Description	Slump, in.	Vebe, sec	Drop table, revolutions	Compacting factor	3/8 in.	1/2 in	3/4 in.	1 in.	1 1/2 in.
Extremely dry	—	32-18	112-56	—	190	170	145	140	130
Very stiff	—	18-10	56-28	0.70	160	145	130	125	125
Stiff	0-1	10- 5	28-14	0.75	135	130	115	115	120
Stiff plastic	1-3	5- 3	14- 7	0.85	108	106	104	106	109
Plastic	3-5	3- 0	<7	0.91	100	100	100	100	100
Flowing	5-7	—	—	0.95	97	98	100	100	100

*Based on tests of non-air-entrained concretes made with a natural sand having a fineness modulus of 2.90 and a rounded gravel, containing some crushed over-size. Maximum sizes used were 3/8 in., 3/4 in., and 1 1/2 in. Values for 1/2 in. and 1 in. are interpolated.
It is assumed, for the purpose of this method, that the multiplication factors shown are appropriate for sands having other fineness moduli. These values are intended as a guide in establishing the first trial mixtures. Further adjustments will be necessary.

4.1.1.4 The fine aggregate has a specific gravity, bulk dry, of 2.64, an absorption of 0.7 percent, and fineness modulus of 2.80.

4.2—Example of computation of proportions

4.2.1—Concrete is required for a precast prestressed girder for a bridge which will be exposed to severe weather with frequent alternations of freezing and thawing. Structural considerations require it to have a design compressive strength of 4000 psi at 28 days. From previous experience in the plant producing these girders, the expected coefficient of variation of strengths is 10 percent. It is further required that no more than one test in ten will fall below the design strength of 4000 psi at 28 days. From Fig. 5 of ACI 214-65, the required average strength at 28 days should be 4000×1.15 or 4600 psi. The size of the section and spacing of prestressing tendons are such that a maximum size aggregate of 1½ in. is indicated and a properly graded No. 4 to 1½ in. coarse aggregate is locally available. Heavy internal and external vibration is available to achieve compaction, enabling the use of very stiff concrete. The dry-rodded weight of the coarse aggregate is found to be 100 lb per cu ft. The proportions may be computed as follows:

4.2.1.1 Since the exposure is a severe one, air-entrained concrete will be used and reference to Table 3.4.1 shows that the water-cement ratio should not exceed 0.50 by weight.

4.2.1.2 From Table 3.4.3, the water-cement ratio required to produce an average 28-day strength of 4600 psi in air-entrained concrete is shown to be about 0.43 by weight. Since this is lower than required for durability considerations, this value of water-cement ratio governs.

4.2.1.3 The approximate quantity of mixing water needed to produce a consistency in the "very stiff" range in air-entrained concrete made with 1½ in. aggregate is found in Table 2.3.1 (c) to be 225 lb per cu yd. In that same table, the desired air content, which in this case will be secured by use of an air-entraining admixture,* is indicated as 4.5 percent for the more plastic mixtures (see note). The note to the table calls attention to the lower air contents entrained in these stiffer mixtures. For this concrete, assume the air content to be 3.0 percent when the suggestions in the note are followed.

4.2.1.4 From Sections 4.2.1.2 and 4.2.1.3, it can be seen that the required cement content is 225/ $0.43 = 523$ lb per cu yd.

4.2.1.5 From Table 3.5.1 (a) it is found that, with a maximum size of aggregate of 1½ in. and a fineness modulus of sand of 2.80, 0.71 cu ft of coarse aggregate, on a dry-rodded basis, would be required in each cubic foot of concrete having a consistency of about 3 to 4-in. slump (plastic).

4.2.1.6 From Table 3.5.1 (b) it is found that for the "very stiff" consistency desired, the amount of coarse aggregate should be 125 percent of that for the "plastic" consistency, or $0.71 \times 1.25 = 0.89$. The quantity in a cubic yard will be $27 \times 0.89 = 24.03$ cu ft which in this case weighs 100×24.03 or 2403 lb.

4.2.1.7 With the quantities of cement, water, coarse aggregate, and air established, the sand content is calculated as follows:

Solid volume of cement	=	$\dfrac{523}{3.15 \times 62.4}$	= 2.66 cu ft
Volume of water	=	$\dfrac{225}{62.4}$	= 3.61 cu ft
Solid volume of coarse aggregate	=	$\dfrac{2403}{2.68 \times 62.4}$	= 14.37 cu ft
Volume of air	=	27×0.030	= 0.81 cu ft
Total solid volume of ingredients except sand			= 21.45 cu ft
Solid volume of sand required	=	$27 - 21.45$	= 5.55 cu ft
Required weight of dry sand	=	$5.55 \times 2.64 \times 62.4$	= 914 lb
Water absorbed by dry aggregates	=	$(914 \times 0.007 = 6.4 \text{ lb}) + (2403 \times 0.005 = 12.0 \text{ lb})$	= **18.4 lb**

4.2.1.8 The estimated batch quantities per cubic yard of concrete are:

Cement	= 523 lb
Water	= 243.4 lb
Sand (dry basis)	= 914 lb
Coarse aggregate (dry basis)	= 2403 lb

4.3—Batch weights for field use

4.3.1—For the sake of convenience in making trial mixture computations, the aggregates have been assumed to be in a dry state. Under field conditions they will generally be moist and the quantities to be batched into the mixer must be adjusted accordingly.

4.3.2†—With the batch weights determined in the example, let it be assumed that tests show the sand to contain 5.0 percent and the coarse aggregate 1.0 percent total moisture. Since the quantity of dry sand required was 914 lb the amount of moist sand to be weighed out must be 914×1.05

*Air-entraining admixture when added at the mixer as fluid should be included as part of the water volume.

†Weights in this section have been calculated to a greater degree of accuracy than is usually required in order to make the comparison shown in Table 4.3.2. For example, the calculated weight of the moist sand (959.7 lb) would usually be taken as 960 lb in the field.

= 959.7 lb. Similarly, the weight of moist coarse aggregate must be 2403 × 1.01 = 2427 lb.

4.3.2.1 The free water on aggregates in excess of their absorption must be considered as part of the mixing water. Since the absorption of sand is 0.7 percent, the amount of free water which it contains is 5.0 − 0.7 = 4.3 percent. The free water on coarse aggregate is 1.0 − 0.5 = 0.5 percent. Therefore, the mixing water contributed by the sand is 0.043 × 914 = 39.3 lb and that contributed by the coarse aggregate is 0.005 × 2403 = 12 lb. The quantity of mixing water to be added, then, is 225 − (39.3 + 12) = 173.7 lb. Table 4.3.2 shows a comparison between the computed batch quantities and those actually to be used in the field for each cubic yard of concrete.

TABLE 4.3.2—COMPARISON BETWEEN COMPUTED BATCH QUANTITIES AND THOSE USED IN THE FIELD

Ingredient	Quantities per cubic yard of concrete, lb	
	Computed	Used in field
Cement	523	523
Net mixing water	225	225
Sand	914 (dry)	959.7 (moist)
Coarse aggregate	2403 (dry)	2427 (moist)
Water absorbed	18.4	
Excess water		—51.3
Total	4083.4	4083.4
Water added at mixer	243.4	173.7

4.3.2.2 The preceding trial mixture computations provide batch quantities for each ingredient of the mixture per cubic yard of concrete. It is seldom desirable or possible to mix concrete in exactly 1 cu yd batches. It is therefore necessary to convert these quantities in proportion to the size batch to be used. Let it be assumed that a 16 cu ft capacity mixer is available. Then to produce a batch of the desired size and maintain the same proportions, the cubic yard field batch weights of all ingredients must be reduced in the ratio 16/27 = 0.593, thus:

Cement	= 0.593 × 523	= 310 lb
Sand (moist)	= 0.593 × 959.7	= 569 lb
Coarse aggregate (moist)	= 0.593 × 2427	= 1439 lb
Water to be added	= 0.593 × 173.7	= 103 lb

4.4—Adjustment of trial mixture

4.4.1—In discussing the estimate of total water requirements given in Table 2.3.1 (c), it was pointed out that in some cases more water might be required than indicated and that, in such cases, the cement factor should be increased to maintain the water-cement ratio, unless otherwise indicated by laboratory tests. This adjustment will be illustrated by assuming that the concrete of the example was found in the field to require 240 lb of net mixing water instead of 225 lb. Consequently, the cement content should be increased from 523 to 240/225 × 523 = 558 lb per cu yd and the batch quantities recomputed accordingly.

4.4.2—It was pointed out also that less water than indicated in Table 2.3.1 (c) may sometimes be required, but it was recommended that no adjustment be made in cement factor, except as indicated by laboratory tests. Nevertheless, some adjustment in batch quantities is necessary to compensate for the loss of volume due to the reduced water. This is done by increasing the solid volume of sand in an amount equal to the volume of the reduction in water. For example, assume that 215 lb are required instead of 225 for the concrete of the example. Then 215/62.4 is substituted for 225/62.4 in computing the volume of water in the batch and the solid volume of sand becomes 5.71 instead of 5.55 cu ft.

The percentage of air in concrete can be measured directly with an air meter (ASTM C 231) or it can be computed from theoretical and measured unit weights in accordance with ASTM test methods listed in the appendix of ACI 211.1-74. For any given set of conditions and materials, the amount of air entrained is roughly proportional to the quantity of air-entraining admixture used. Increasing the cement content or the fine fraction of the sand, decreasing slump, or raising the temperature of the concrete usually decreases the amount of air entrained for a given amount of admixture. The grading and particle shape of aggregate also have an effect on the amount of air entrained. The job mixture should not be adjusted for minor fluctuations in water-cement ratio or air content. A variation in water-cement ratio of ± 0.02, resulting from maintenance of a constant slump, is considered normal. A variation of ± 1 percent in air content is also considered normal. This variation in air content will be smaller in the drier mixtures.

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6. Thaulow, Sven, *Concrete Proportioning*, Norsk Cementforening, Oslo, 1955. (in Norwegian with English Summary)
7. Klieger, Paul, "Some Aspects of Durability and Volume Change of Concrete for Prestressing," *Journal*, PCA Research and Development Laboratories, V. 2, No. 3, Sept. 1960, pp. 2-12 (See Table 3).
8. *Methods of Testing Concrete* (B.S. 1881:1952), British Standards Institution, London, 1952, 61 pp.

APPENDIX 1—METRIC SYSTEM ADAPTATION*

A1.1—General

Procedures outlined in this recommended practice have been presented using United States customary units of measurement. The principles are equally applicable in the metric system with proper adaptation of units. This appendix provides all of the information necessary to apply the proportioning procedure using SI (metric) measurements. Table A1.1 gives relevant conversion factors. A numerical example is presented in Appendix 2.

A1.2—Tables

For convenience of reference all tables, except Tables A1.1 and A3.8, are presented in Appendix 2 near their reference in the design example. They have the same number previously used except that the designation "A2" is prefixed. All tables have been converted and reproduced. Descriptive portions are included only where use of the metric system requires a change in a procedure or formula. To the extent practicable, conversions to metric units have been made in such a way that values are realistic in terms of usual practice and significance of numbers. For example, aggregate and sieve sizes in the metric tables are ones commonly used in Europe. Thus, there is not always a precise mathematical correspondence between U. S. customary and metric values in corresponding tables.

TABLE A1.1—CONVERSION FACTORS, BRITISH TO METRIC UNITS*

Quantity	British (U.S. customary) unit	SI† (Metric) unit	Conversion factor (Ratio: British/SI)
Length	inch (in.)	centimeter (cm)	2.540
	inch (in.)	millimeter (mm)	25.40
Volume	cubic foot (ft^3)	cubic meter (m^3)	0.02832
	cubic yard (yd^3)	cubic meter (m^3)	0.7646
Mass	pound (lb)	kilogram (kg)	0.4536
Stress	pounds per square inch (psi)	kilograms force per square centimeter (kgf/cm^2)	0.0703
Density	pounds per cubic foot (lb/ft^3)	kilograms per cubic meter (kg/m^3)	16.02
	pounds per cubic yard (lb/yd^3)	kilograms per cubic meter (kg/m^3)	0.5933
Temperature	degrees Fahrenheit (F)	degrees Centigrade (C)	‡

*Gives names (and symbols) of measurement units in the British (U.S. customary) system as used in the body of this report and in the S.I. (metric) system, along with multipliers for converting the former to the latter. From "ASTM Metric Practice Guide" (2nd Edition, 1966).
†Systéme International d'Unités
‡$C = (F - 32)/1.8$.

A1.3—Steps in calculating proportions

The methods for arriving at quantities of ingredients for a unit volume of concrete are essentially the same when metric units are employed as when U. S. customary units are employed. The main difference is that the unit volume of concrete becomes the cubic meter and numerical values must be taken from the proper A2 table instead of the one referred to in the text.

A1.3.1 *Step 1*—Select consistency—See Tables A2.2.3.1 (a) and A.2.2.3.1 (b).

*Information in this appendix is based on or taken directly from Appendix 1—Metric System Adaptation, ACI 211.1-74.

TABLE A2.2.3.1(a)—COMPARISON OF CONSISTENCY MEASUREMENTS BY VARIOUS METHODS

Consistency description	Slump, cm	Vebe, sec	Compacting factor, average	Drop table, revolutions
Extremely dry	—	32 to 18	—	112 to 56
Very stiff	—	18 to 10	0.70	56 to 28
Stiff	0 to 3	10 to 5	0.75	28 to 14
Stiff plastic	3 to 8	5 to 3	0.85	14 to 7
Plastic	8 to 13	3 to 0	0.90	<7
Flowing	13 to 18	—	0.95	—

TABLE A2.2.3.1(c)—APPROXIMATE MIXING WATER REQUIREMENTS FOR DIFFERENT CONSISTENCIES AND MAXIMUM SIZES OF AGGREGATES*

Description	Consistency: Slump, cm	Consistency: Vebe, sec	Consistency: Drop table, revolutions	Consistency: Compacting factor	Relative water content, percent	Water, kg/m³ of concrete for indicated maximum sizes of coarse aggregate in mm: 10	12.5	20	25	40
Non-air-entrained concrete										
Extremely dry	—	32-18	112-56	—	78	180	170	160	150	140
Very stiff	—	18-10	56-28	0.70	83	185	185	170	160	150
Stiff	0- 3	10- 5	28-14	0.75	88	200	195	180	170	155
Stiff plastic	3- 8	5- 3	14- 7	0.85	92	205	200	185	180	160
Plastic	8-13	3- 0	<7	0.91	100	225	215	200	195	175
Flowing	13-18	—	—	0.95	106	240	230	210	205	185
Approximate amount of entrapped air in non-air-entrained concrete, percent						3	2.5	2	1.5	1
Air-entrained concrete										
Extremely dry	—	32-18	112-56	—	78	155	150	140	135	125
Very stiff	—	18-10	56-28	0.70	83	170	160	150	140	135
Stiff	0- 3	10- 5	28-14	0.75	88	175	170	160	150	140
Stiff plastic	3- 8	5- 3	14- 7	0.85	92	180	175	165	160	145
Plastic	8-13	3- 0	<7	0.91	100	200	190	180	175	160
Flowing	13-18	—	—	0.95	106	215	205	190	185	170
Recommended average total air content, percent†						8	7	6	5	4.5

*These quantities of mixing water are for use in computing cement factors for trial batches. They are for reasonably well-shaped angular coarse aggregates graded within limits of accepted specifications.

If *more* water is required than shown, the cement factor, estimated from these quantities, should be increased to maintain desired water-cement ratio, except as otherwise indicated by laboratory tests for strength.

If *less* water is required than shown, the cement factor, estimated from these quantities, *should not* be decreased except as indicated by laboratory tests for strength.

†For consistencies below 3 cm slump, the volume of air entrained by either an air-entraining cement or the usual amount of air-entraining admixture used for more plastic mixtures may be significantly lower than those shown. For these mixtures, it is recommended that the air content resulting from the use of air-entraining cement or the usual amount of air-entraining admixture per unit of cement for more plastic mixtures be accepted as adequate for insuring durability. In the absence of such information for a particular air-entraining admixture, the amount to use per unit of cement can be determined on a trial mix having a slump in the 8 to 10 cm range, or by determining the amount needed to obtain 19 ± 3 percent air in mortar prepared in accordance with ASTM C 185.[7]

TABLE A2.2.3.1(b)—APPROXIMATE RELATIVE WATER CONTENT FOR DIFFERENT CONSISTENCIES

Consistency description	Approximate relative water content, percent	
	Thaulow[6]	Table 5.3.3 ACI 211.1-74
Extremely dry	78	—
Very stiff	83	—
Stiff	88	—
Stiff plastic	93	92
Plastic	100	100
Flowing	108	106

A1.3.2 *Step 2*—Select maximum size of aggregate.

A1.3.3 *Step 3*—Determine w/c, by weight, needed for durability (Table A2.3.4.1) and for strength (Table A2.3.4.3). The lower w/c ratio governs and should be used in subsequent calculations.

A1.3.4 *Step 4*—Determine the approximate quantity of mixing water needed for the consistency and maximum aggregate size and the percentage of entrapped or entrained air from Table A2.2.3.1(c).

A1.3.5 *Step 5*—Calculate the cement content.

A1.3.6 *Step 6*—Determine coarse aggregate content. The dry weight of coarse aggregate required for a cubic meter of concrete is equal to the value from Table A2.3.5.1(a) multiplied by the dry-rodded unit weight of the aggregate in kilograms per cubic meter multiplied by the appropriate factor from Table A2.3.5.1(b).

A1.3.7 *Step 7*—Determine the fine aggregate content. The absolute volume of the fine aggregate is obtained by (a) calculating the absolute volumes of the cement, water, coarse aggregate and air in cubic meters, and (b) subtracting the sum of these absolute volumes from 1.000 cubic meters. The required weight of fine aggregate, kg, is then calculated by multiplying its absolute volume by its specific gravity and the product by 1000.

A1.3.8 *Step 8*—In the field, necessary corrections must be made for the water absorbed by the aggregates if they are dry, or by the excess water provided by the aggregates if they are moist.

APPENDIX 2—EXAMPLE PROBLEM IN METRIC SYSTEM

A2.1—Description of problem

The example presented in Section 4.2 will be solved here using metric units of measure. The required compressive strength will be 325 kgf/cm² at 28 days. The size of the section and spacing of prestressing tendons are such that a maximum size aggregate of 40 mm is indicated and a properly graded coarse aggregate (4.75 mm to 40 mm) is locally available. The dry-rodded weight of the coarse aggregate is 1602 kg/m³. The coarse aggregate has a specific gravity, bulk dry, of 2.68 and an absorption of 0.5 percent. The fine aggregate has a specific gravity, bulk dry, of 2.64, an absorption of 0.7 percent, and a fineness modulus of 2.80. The prestressed bridge girder will be exposed to severe weather with frequent alternations of freezing and thawing. Heavy internal and external vibration is available to achieve compaction, enabling the use of concrete having a very stiff consistency. The proportions may be computed as follows:

A2.1.1—Since the exposure is a severe one, air-entrained concrete will be used and reference to Table A2.3.4.1 shows that the water-cement ratio should not exceed 0.50, by weight.

A2.1.2—From Table A2.3.4.3, the water-cement ratio required to produce an average 28 day strength of 325 kgf/cm² in air-entrained concrete is shown to be about 0.43, by weight. Since this

TABLE A2.3.4.1—MAXIMUM PERMISSIBLE WATER-CEMENT RATIOS FOR CONCRETE IN SEVERE EXPOSURES (METRIC)*

Type of structure	Structure wet continuously or frequently and exposed to freezing and thawing†	Structure exposed to seawater or sulfates
Thin sections (railings, curbs, sills, ledges, ornamental work) and sections with less than 3 cm cover over steel	0.45	0.40‡
All other structures	0.50	0.45‡

*Based on the report of ACI Committee 201, "Durability of Concrete in Service," previously cited.
†Concrete should also be air-entrained.
‡If sulfate resisting cement (Type II or Type V of ASTM C 150) is used, permissible water-cement ratio may be increased by 0.05.

TABLE A2.3.4.3—RELATIONSHIPS BETWEEN WATER-CEMENT RATIO AND COMPRESSIVE STRENGTH OF CONCRETE (METRIC)

Compressive strength at 28 days, kgf/cm²*	Water-cement ratio, by weight	
	Non-air-entrained concrete	Air-entrained concrete
500	0.33	—
450	0.38	—
400	0.43	0.34
350	0.48	0.40
300	0.55	0.46
250	0.62	0.53
200	0.70	0.61
150	0.80	0.71

*Values are estimated average strengths for concrete containing not more than the percentage of air shown in Table A2.2.3.1(c). For a constant water-cement ratio, the strength of concrete is reduced as the air content is increased. See Section 3.4.5.

Strength is based on 15 x 30 cm cylinders moist-cured 28 days at 23 ± 1.7 C in accordance with Section 9(b) of ASTM C 31 for Making and Curing Concrete Compression and Flexure Test Specimens in the Field. Cube strengths will be higher by approximately 20 percent.

Relationship assumes maximum size of aggregate about 20 to 30 mm; for a given source, strength produced by a given water-cement ratio will increase as maximum size decreases.

is lower than required for durability considerations, this value of water-cement ratio governs.

A2.1.3—The approximate quantity of mixing water needed to produce a consistency in the "very stiff" range in air-entrained concrete made with 40 mm aggregate is found in Table A2.2.3.1 (c) to be 135 kg/m³. In that same table, the desired air content, which in this case will be secured by use of an air-entraining admixture,* is indicated as 4.5 percent for the more plastic mixtures (see note). The note to the table calls attention to the lower air contents entrained in these stiffer mixtures. For this concrete, assume the air content to be 3.0 percent when the suggestions in the note are followed.

A2.1.4—From Sections A2.1.2 and A2.1.3, it can be seen that the required cement content is 135/0.43 = 314 kg/m³.

A2.1.5—From Table A2.3.5.1 (a) it is found that, with a maximum size of aggregate of 40 mm and a fineness modulus of sand of 2.80, 0.72 m³ of coarse aggregate, on a dry-rodded basis, would be required in each cubic meter of concrete having a consistency of about 8 to 10-cm slump (plastic).

A2.1.6—From Table A2.3.5.1 (b) it is found that for the "very stiff" consistency desired, the amount of coarse aggregate should be 125 percent of that for the "plastic" consistency, or $0.72 \times 1.25 = 0.90$. The weight of the coarse aggregate will be $0.90 \times 1602 = 1442$ kg.

A2.1.7—With the quantities of cement, water, coarse aggregate, and air established, the sand content is calculated as follows:

Solid volume of cement	=	$\frac{314}{3.15 \times 1000}$	= 0.100 m³
Volume of water	=	$\frac{135}{1000}$	= 0.135 m³
Solid volume of coarse aggregate	=	$\frac{1442}{2.68 \times 1000}$	= 0.538 m³
Volume of air	=	0.03×1.000	= 0.030 m³
Total volume of ingredients except sand	=		0.803 m³
Solid volume of sand required	=	$1.000 - 0.803$	= 0.197 m³
Required weight of dry sand	=	$0.197 \times 2.64 \times 1000$	= 520 kg
Water absorbed by aggregates	=	$(520 \times 0.007 = 3.6\text{ kg}) + (1442 \times 0.005 = 7.2\text{ kg})$	= 10.8 kg

The estimated batch quantities per cubic meter of concrete are:

Cement	=	314 kg
Water	=	145.8 kg
Sand (dry)	=	520 kg
Coarse aggregate (dry)	=	1442 kg

*Air-entraining admixture when added at the mixer as fluid should be included as part of the water volume.

TABLE A2.3.5.1(a)—VOLUME OF COARSE AGGREGATE PER UNIT OF VOLUME OF CONCRETE (METRIC) OF PLASTIC CONSISTENCY (8-13 CM SLUMP)

Maximum size of aggregate, mm	Volume of dry-rodded coarse aggregate* per unit volume of concrete for different fineness moduli† of sand			
	2.40	2.60	2.80	3.00
10	0.50	0.48	0.46	0.44
12.5	0.59	0.57	0.55	0.53
20	0.66	0.64	0.62	0.60
25	0.71	0.69	0.67	0.65
40	0.76	0.74	0.72	0.70

*Volumes are based on aggregates in dry-rodded condition as described in ASTM C 29 for Unit Weight of Aggregate.

These volumes are selected from empirical relationships to produce concrete with a degree of workability suitable for usual reinforced construction.

†Fineness modulus of sand = sum of ratios (cumulative) retained on sieves with square openings of 0.149, 0.297, 0.595, 1.19, 2.38, and 4.76 mm.

TABLE A2.3.5.1(b)—VOLUME OF COARSE AGGREGATE PER UNIT VOLUME OF CONCRETE FOR DIFFERENT CONSISTENCIES*

Consistency					Volume of dry-rodded coarse aggregate per unit volume of concrete for maximum size of aggregate shown (expressed as a percentage of the values shown in Table A2.3.5.1 (a)				
Description	Slump, cm	Vebe, sec	Drop table, revolutions	Compacting factor	10 mm	12.5 mm	20 mm	25 mm	40 mm
Extremely dry	—	32-18	112-56	—	190	170	145	140	130
Very stiff	—	18-10	56-28	0.70	160	145	130	125	125
Stiff	0- 3	10- 5	28-14	0.75	135	130	115	115	120
Stiff plastic	3- 8	5- 3	14- 7	0.85	108	106	104	106	109
Plastic	8-13	3- 0	<7	0.91	100	100	100	100	100
Flowing	13-18	—	—	0.95	97	98	100	100	100

*Based on tests of non-air-entrained concretes made with a natural sand having a fineness modulus of 2.90 and a rounded gravel, containing some crushed over-size. Maximum sizes used were 10, 20, and 40 mm. Values for 12.5 mm and 25 mm are interpolated.

It is assumed, for the purpose of this method, that the multiplication factors shown are appropriate for sands having other fineness moduli. These values are intended as a guide in establishing the first trial mixtures. Further adjustments will be necessary.

A2.2—Batch weights for field use

A2.2.1—For the sake of convenience in making trial mixture computations, the aggregates have been assumed to be in a dry state. Under field conditions they will generally be moist and the quantities to be batched into the mixer must be adjusted accordingly.

A2.2.2—With the batch weights determined in the example, let it be assumed that tests show the sand to contain 5.0 percent and the coarse aggregate 1.0 percent total moisture. Since the quantity of dry sand required was 520 kg, the amount of moist sand to be weighed out must be $520 \times 1.05 = 546$ kg. Similarly, the weight of moist coarse aggregate must be $1442 \times 1.01 = 1456.4$ kg.

A2.2.2.1 The free water on aggregates in excess of their absorption must be considered as part of the mixing water. Since the absorption of sand is 0.7 percent, the amount of free water which it contains is $5.0 - 0.7 = 4.3$ percent. The free water on coarse aggregate is $1.0 - 0.5 = 0.5$ percent. Therefore, the mixing water contributed by the sand is $520 \times 0.043 = 22.4$ kg and that contributed by the coarse aggregate is $1442 \times 0.005 = 7.2$ kg. The quantity of mixing water to be added, then, is $135.0 - (22.4 + 7.2) = 105.4$ kg. Table A2.4.3.2 shows a comparison between the computed batch quantities and those actually to be used in the field for each cubic meter of concrete.

A2.2.2.2 The preceding trial mixture computations provide batch quantities for each ingredient of the mixture per cubic meter of concrete. It is seldom desirable or possible to mix concrete in exactly 1 cubic meter batches. However, conversion to other mixer capacities is easily made.

A2.3—Adjustment of trial mixture

A2.3.1—In discussing the estimate of total water requirements given in Table A2.2.3.1 (c), it was pointed out that in some cases more water might be required than indicated and that, in such cases, the cement factor should be increased to maintain the water-cement ratio, unless otherwise indicated by laboratory tests. This adjustment will be illustrated by assuming that the concrete of the example was found in the field to require 145 kg of water instead of 135 kg. Consequently, the cement content should be increased from 314 to $145/135 \times 314 = 337.3$ kg and the batch quantities recomputed accordingly.

A2.3.2—It was pointed out also that less water than indicated in Table A2.2.3.1 (c) may sometimes be required, but it was recommended that no adjustment be made in cement factor, except as indicated by laboratory tests. Nevertheless, some adjustment in batch quantities is necessary to compensate for the loss of volume due to the reduced water. This is done by increasing the solid volume of sand in an amount equal to the volume of the reduction in water. For example, assume that 130 kg are required instead of 135 for the concrete of the example. Then 130/1000 is substituted for 135/1000 in computing the volume of water in the batch and the solid volume of sand becomes 0.202 instead of 0.197 m^3.

TABLE A2.4.3.2—COMPARISON BETWEEN COMPUTED BATCH QUANTITIES AND THOSE USED IN THE FIELD

Ingredient	Quantities per cubic meter of concrete, kg	
	Computed	Used in field
Cement	314	314
Net water	135	135
Sand	520 (dry)	546 (moist)
Coarse aggregate	1442 (dry)	1456.4 (moist)
Water absorbed	10.8	
Excess water		—29.6
Total	2421.8	2421.8
Water added at mixer	145.8	105.4

A2.3.3—The percentage of air in concrete can be measured directly with an air meter (ASTM C 231) or it can be computed from the theoretical and measured unit weights in accordance with ASTM test methods listed in the appendix of ACI 211.1-74. For any given set of conditions and materials, the amount of air entrained is roughly proportional to the quantity of air-entraining admixture used. Increasing the cement content or the fine fraction of the sand, decreasing slump, or raising the temperature of the concrete usually decreases the amount of air entrained for a given amount of admixture. The grading and particle shape of aggregate also have an effect on the amount of air entrained. The job mixture should not be adjusted for minor fluctuations in water-cement ratio or air content. A variation in water-cement ratio of $\pm$ 0.02, resulting from maintenance of a constant slump, is considered normal. A variation of $\pm$ 1 percent in air content is also considered normal. This variation in air content will be smaller in the drier mixtures.

APPENDIX 3—LABORATORY TESTS

A3.1—General

As stated in the introduction, selection of concrete mixture proportions can be accomplished most effectively from results of laboratory tests which determine basic physical properties of materials to be used, establish relationships between water-cement ratio, air content, cement content, and strength, and which furnish information on the workability characteristics of various combinations of ingredient materials. The extent of investigation desirable for any given job will depend on its size and importance and service conditions involved. Details of the laboratory program will also vary, depending on facilities available and on individual preferences.

A3.2—Physical properties of cement

A3.2.1—Physical and chemical characteristics of cement influence the properties of hardened concrete. However, the only property of cement directly concerned in computation of concrete mix proportions is specific gravity. As stated, the specific gravity of cement may be assumed to be 3.15 without introducing appreciable error in mix computations.

A3.2.2—A sample of cement should be obtained from the mill which will supply the job, or preferably from the job itself. The sample should be ample for tests contemplated with a liberal margin for additional tests that might later be considered desirable. Cement samples should be shipped in airtight containers, or at least in moisture-proof packages.

A3.3—Properties of aggregate

A3.3.1—Sieve analysis, specific gravity, absorption, and moisture content of both fine and coarse aggregate and dry-rodded unit weight of coarse aggregate are essential physical properties required for mixture computations. Other tests which may be desirable for large or special types of work include petrographic examination and tests for chemical reactivity; tests for soundness, durability, resistance to abrasion, and for various deleterious substances. All such tests yield information of value in judging the ultimate quality of concrete and in selecting appropriate proportions.

A3.3.2—Aggregate gradation or particle size distribution is a major factor in controlling unit water requirement, proportion of coarse aggregate to sand, and cement content of concrete mixtures for a given degree of workability. Numerous "ideal" aggregate grading curves have been proposed, but a universally accepted standard has not been developed. Experience and individual judgment must continue to play important roles in determining acceptable aggregate gradings. Additional workability realized by use of air entrainment permits, to some extent, the use of less restrictive aggregate gradations.

A3.3.3—Undesirable sand grading may be corrected to desired particle size distribution by: (1) separation of the sand into two or more size fractions and recombining in suitable proportions; (2) increasing or decreasing the quantity of certain sizes to balance the grading; (3) reducing excess coarse material by grinding; or (4) by the addition of manufactured sand. Undersirable coarse aggregate gradings may be corrected by: (1) crushing excess coarser fractions; (2) wasting excess material in other fractions; (3) supplementing deficient sizes from other sources; or (4) a combination of these methods. To the extent that grading limitations and economy in use of cement permit, the proportions of various sizes of coarse aggregate should be held closely to the grading of available materials. Whatever processing is done in the laboratory should be practical from a standpoint of economy and job operation. Samples of aggregates for concrete mixture tests should be representative of aggregate selected for use in the work. For laboratory tests, the coarse aggregates should be cleanly separated into required size fractions to provide for uniform control of mixture proportions.

A3.3.4—The particle shape and texture of both fine and coarse aggregate also influence the mixing water requirement of concrete. Void content of compacted dry fine or coarse aggregate can be used as an indicator of angularity. Void contents of more than 40 percent in conventionally graded aggregates indicate angular harsh material which will probably require more mixing water than given in Table 2.3.1(c). Conversely rounded aggregates with voids below 35 percent will probably need less water.

A3.4—Concrete mixture tests

A3.4.1—The values listed in the tables may be used for establishing a preliminary trial mixture. However, they are based on averages obtained from a large number of tests and do not necessarily apply exactly to materials being used on a particular job. If facilities are available, therefore, it is advisable to make a series of concrete tests to establish the relationships needed for selection of appropriate proportions based on the materials actually to be used.

A3.4.2—Air-entrained concrete or concrete with no measurable slump must be machine mixed. Before mixing the first batch the laboratory mixer should be "buttered" as described in ASTM C 192 because a clean mixer retains a percentage of mortar. Similarly, any processing of materials in the laboratory should simulate as closely as practicable corresponding treatment in the field. Adjustments of the preliminary trial mixture will almost always be necessary. Furthermore, it should not be expected that field results will check exactly with laboratory results. An adjustment of the selected trial mixture on the job is usually necessary.

A3.4.3—Alternative aggregate sources and different aggregate gradings, different types and brands of cement, different admixtures, different maximum sizes of aggregate, and considerations of concrete durability, volume change, temperature rise, and thermal properties are some of the variables that may require a more extensive program.

A3.5—Specifications and test methods

A3.5.1 — Appropriate specifications and test methods for the various ingredients in concrete and for the freshly-mixed and hardened concrete are published by the American Society for Testing and Materials, the American Association of State Highway Officials and various Federal and State agencies. A list of useful test methods is shown in the appendix to ACI 211.1-74.

A3.6—Equipment and techniques for measuring consistency

A3.6.1—The following is a more detailed description of the equipment and techniques involved in the three methods for measuring consistency described in Section 2.2.

A3.7—Vebe apparatus

A3.7.1—The Vebe apparatus is manufactured by Dynapac Maskin AB (earlier Vibro-Verken), Box 1103, S-171 22 Solna, Sweden. It is distributed in the United States by Dynapac Manufacturing Inc., Stanhope, N.J. 07874.*

A3.7.2—The apparatus consists of a heavy base, resting on three rubber feet, a vibrator table supported on rubber shock absorbers, a vibromotor with rotating eccentric weight, a cylindrical metal to hold the concrete sample (approximate inside dimensions: 9½ in. in diameter and 7¾ in. high), a metal slump cone (ASTM C 143), a funnel for filling slump cone, a swivel arm holding a graduated metal rod and clear plastic disk (diameter of disk slightly less than diameter of sample pot). The vibrator table is 15 in. in length, 10¼ in. in width and 12 in. in height. The overall width, with the disk swung away from the pot, is 26½ in. The overall height above floor level from the top edge of the funnel used to fill the slump cone is about 28 in. The total weight of the equipment is about 210 lb. Fig. 2.2.3 shows the apparatus mounted on a concrete pedestal about 15 in. in height.

A3.7.3—As shown in Fig. 2.2.3, the sample of concrete is compacted in the slump cone, the top struck off, the cone removed, and the slump measured, as per ASTM C 143. The swivel arm is then moved into position with the plastic disk and graduated rod resting on top of the concrete sample. The vibrator is switched on and the time in seconds to deform the cone into a cylinder, at which stage the whole face of the plastic disk is in contact with the concrete, is determined. This time in seconds is used as a measure of the consistency of the concrete.

A3.8—Compacting factor apparatus

A3.8.1—The compacting factor test is a British standard. The equipment and technique of operation is described in Part 3 of British Standard 1881: 1952, Methods of Testing Concrete, British Standards Institution. The equipment can be purchased in Britain from Winget, Ltd., Rochester, Kent, England. It is relatively simple to construct in accordance with the requirements of B. S. 1881: 1952. Studies utilizing a compacting factor apparatus built in the United States in accordance with these requirements are reported in U. S. Army Engineer Waterways Experiment Station Technical Report No. 6-598, "Investigation of Partially Compacted Weight of Concrete as a Measure of Workability," April, 1962.

A3.8.2—The essential dimensions of the apparatus (Fig. 2.2.4) are shown in Table A3.8.

A3.8.3—The hoppers and cylinder molds can be made of steel provided the inside surfaces are smooth and joints are smooth and flush. The lower ends of the hoppers are closed with tightly fitting,

*The Vebe apparatus and the compacting factor equipment are also available from Soiltest, Inc., 2205 Lee St., Evanston, Ill. 60202.

TABLE A3.8—DIMENSIONS OF COMPACTING FACTOR APPARATUS

Detail (See Fig. 2.2.4)	Dimensions, in.
A—Upper hopper	
Top inside diameter	10
Bottom inside diameter	5
Inside height	11
B—Lower hopper	
Top inside diameter	9
Bottom inside diameter	5
Inside height	9
C—Cylinder	
Inside diameter	6
Inside height	12
Distance between bottom of upper hopper and top of lower hopper	8
Distance between bottom of lower hopper and top of cylinder	8

quick release, hinged trap doors. Metal plate ⅛ in. thick is suitable for these trap doors. The frame is rigidly constructed using steel angles or channels. The hoppers and the cylinder are easily detachable from the frame. Accessory equipment required includes two steel trowels, a hand scoop about 6 in. long, a steel rod ⅝ in. in diameter and 24 in. long, rounded at one end, and a scale or balance having a capacity of about 45 to 50 lb and with a least scale division of about 0.10 lb.

A3.8.4—The sample of concrete is placed gently in the upper hopper, using the hand scoop, level with the top of the hopper (the precise quantity is not important). The trap door is opened, permitting the concrete to fall into the lower hopper. The trap door of the lower hopper is then opened, permitting the concrete to fall into the cylinder. Without further consolidation the mold is struck off, weighed and the contents then discarded. The cylinder mold is then refilled with concrete which is thoroughly compacted by vibration, or otherwise, and again weighed. The ratio of the weight of partially compacted concrete to the weight of the fully compacted concrete is called the compacting factor.

A3.9—Thaulow drop table

A3.9.1—The Thaulow drop table, developed by Sven Thaulow, is part of a collection of equipment assembled for field testing of concrete. It is described in Reference 5, a résumé of which was published in the ACI News Letter referred to in the same reference. The equipment is available commercially through Ingeniorforretningen Atlas A/S, Postboks 198, Oslo 7, Norway.

A3.9.2—The sample container has a volume of 10 litres and is made of hardened aluminum alloy or stainless steel (latest model) having an inside diameter of 9½ in. and an inside height of 9¼ in. A 5-litre mark is grooved on the inside surface of the container. The slump cone meets the requirements of ASTM C 143, with the funnel adaptor shown as an aid in filling and compaction. The drop table, to which the container is clamped, is operated by a hand crank, each revolution of which drops the table 0.394 in. four times. The number of revolutions are indicated on an attached revolution counter. The triangular concrete base, 3 in. thick and 16 in. on each side, provides stability for the unit.

A3.9.3—The slump cone is filled and compacted in accordance with ASTM C 143, after which additional concrete is added in the funnel adaptor and the whole assembly subjected to 15 drops, turning the handle at the rate of 1½ - 2 revolutions per sec. After strike-off and removal of the cone, the number of revolutions required to deform the cone (5½ litres) to a 5-litre cylinder with a half-litre topping, i.e., when the entire periphery of the concrete is at the 5-litre mark, is a measure of the consistency of the concrete.

ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction, and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be incorporated directly into the Project Documents.

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Guide for Use of Admixtures in Concrete

Reported by ACI Committee 212

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This guide outlines the purposes of and factors to be considered in the use of admixtures. One chapter gives information on the preparation and batching of admixtures. Other chapters give information on the properties, effects, and use of various classes of admixtures. Information is included on: (1) air-entraining admixtures; (2) accelerators; (3) water-reducing and set-controlling admixtures; (4) finely divided mineral admixtures, including relatively inert powders, cementitious materials, and pozzolans; and (5) miscellaneous admixtures, including gas-forming, grouting, expansion-producing, bonding, coloring, flocculating, fungicidal, germicidal, insecticidal, dampproofing, and permeability-reducing admixtures; admixtures to reduce alkali-aggregate reaction expansion; and corrosion inhibitors.

Keywords: accelerating agents; admixtures; air-entraining agents; alkali-aggregate reactions; aluminum powder; bactericides; batching; bonding; calcium chlorides; colors (materials); concrete durability; concretes; corrosion inhibitors; dispensers; expanding agents; flocculating; freeze-thaw durability; fungicides; gas-forming agents; grouting; hydroxylated carboxylic acids; insecticides; lignin and derivatives; lignosulfonates; mineral admixtures; mix proportioning; permeability reducing admixtures; pozzolans; quality control; retardants; setting (hardening); shrinkage; strength; sulfate resistance; temperature rise (in concrete); vinsol resin; volume change; water-reducing agents; waterproofing admixtures.

FOREWORD

ACI Committee 212, Admixtures for Concrete, was organized in 1943. It has functioned in close, but informal, liaison with the Highway Research Board Committee MC-B5 (now A 2E05) on Admixtures and Subcommittee III-h on Specifications and Methods of Testing Admixtures of ASTM Committee C-9 on Concrete and Concrete Aggregates. Symposia, bibliographies, reports of research, and the state-of-the-art reviews sponsored by all three groups have been studied and used, with benefit, by the other two. Recent reviews covering significant portions of the area of interest of Committee 212 were distributed in 1970 as Part IV (V. IV) of the Proceedings of the Fifth International Symposium on the Chemistry of Cement, entitled "Admixtures and Spe-

*Deceased

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cial Cements;" and as "The Science of Admixtures," proceedings of a symposium organized jointly by the Concrete Society and the Cement Admixture Association, London, Nov. 6, 1969 (published in 1970 by the Concrete Society, London, 70 pp.).

The mission of preparing an ACI guide for the use of admixtures in concrete was assigned Committee 212 in 1964. The major work was accomplished between 1964 and 1969 under the chairmanship of Robert F. Adams. The various sections of this guide were brought to completion by task groups under the chairmanship of Robert F. Adams, H. Bobbitt Aikin, H. C. Fischer, Paul Klieger, K. R. Lauer, Richard C. Mielenz, and Melville E. Prior.

CONTENTS

CHAPTER 1 – INTRODUCTION AND GENERAL INFORMATION

1.1 — Introduction

An admixture is: "A material other than water, aggregates, and hydraulic cement that is used as an ingredient of concrete or mortar and is added to the batch immediately before or during its mixing."[1] Committee 212 deals with all commonly recognized admixtures except those whose use results in a special type of concrete assigned to another ACI committee. At present, these exclusions are: insulating and cellular concretes (ACI Committee 523); fiber reinforced concrete (ACI Committee 544); and polymer concretes (ACI Committee 548). Expansive cement concretes are no longer discussed by Committee 212 since they are now covered in the report of ACI Committee 223.[2]

1.1.1 — Admixtures may be used to modify the properties of the concrete in such a way as to make it more suitable for the work at hand or for economy. An admixture should be employed only after appropriate evaluation of its effects shows it to be desirable for use in the particular concrete and under conditions of use intended. Admixtures should be required to conform with applicable ASTM or other relevant specifications. In using any admixture, careful attention should be given to the instructions provided by the manufacturer of the product.

1.1.2 — Selection, evaluation, and methods of addition of admixtures are discussed in this guide with the objective of enabling the user to develop information necessary for selection and proper use.

1.1.3 — Admixtures can be used for the following purposes:

1. To increase workability without increasing water content or to decrease the water content at the same workability
2. To accelerate the rate of strength development at early ages
3. To increase the strength
4. To either retard or accelerate initial setting
5. To retard or reduce heat evolution
6. To modify rate of or capacity for bleeding, or both
7. To increase durability or resistance to severe conditions of exposure including application of ice-removal salts.
8. To control expansion caused by the reaction of alkalies with certain aggregate constituents.
9. To decrease capillary flow of water
10. To decrease the permeability to liquids
11. To produce cellular concrete
12. To improve penetration and pumpability
13. To reduce segregation particularly in grout mixtures seperation during mixture of aggregates
14. To reduce or prevent settlement, or to create slight expansion, in concrete or mortar used for filling block-outs or other openings in concrete structures, and in grout for seating machinery, columns or girders, or for filling post-tensioning cable ducts or voids in preplaced aggregate
15. To increase bond of concrete to steel
16. To increase bond between old and new concrete
17. To produce colored concrete or mortar
18. To produce fungicidal, germicidal, and insecticidal properties in concrete or mortars
19. To inhibit corrosion of embedded corrodible metal
20. To decrease the unit cost of concrete

1.1.4 — The foregoing list of 20 important purposes for which admixtures are used is, in effect, a functional classification. For more detailed information on each, reference may be made to the third report of ACI Committee 212, "Admixtures for Concrete."[3] Although changes in admixture technology have occurred since that report was published in 1963, (as shown by many statements in the present report), it still contains much useful reference information.

1.2 — Classification of admixtures

Admixtures for concrete may be classified by function as above, or by composition. The 1959 American Society for Testing and Materials Symposium[4] on the effect of water-reducing admixtures and set-retarding admixtures on properties of concrete used a classification based on composition because the bulk of these products in use were of two types or modifications thereof. Shortly thereafter, however, Subcommittee III-h on Admixtures of ASTM Committee C-9, started development of what has come to be ASTM C 494, Chemical Admixtures for Concrete.[5] This standard classifies certain chemical admixtures in terms of function:

Type A — Water-reducing admixtures

Type B — Retarding admixtures

Type C — Accelerating admixtures

Type D — Water-reducing and retarding admixtures

Type E — Water-reducing and accelerating admixtures

1.3 — Specifications for admixtures

The following specifications are most widely used and they cover those types or classes that make up the bulk of current production of admixtures:

ASTM D 98[6]	Calcium Chloride
ASTM C 260[7]	Air-Entraining Admixtures
AASHO M 154[8]	Air-Entraining Admixtures
CRD-C 13[9]	Air-Entraining Admixtures
ASTM C 618[10]	Fly Ash and Raw or Calcined Natural Pozzolans
Federal Specification SS-P-570b[11]	Pozzolans
CRD-C 262[12]	Pozzolan
ASTM C 494[5]	Chemical Admixtures
CRD-C 87[13]	Chemical Admixtures
USBR[14]	Water-Reducing Set-Controlling Admixtures

1.4 — Sampling

Samples for testing and inspection should be obtained by procedures prescribed for the respective types of materials in applicable specifications, such as those listed in Section 1.3. Samples for use in qualifying tests should be obtained from plant production and from previously unopened packages or containers, or from fresh bulk shipments. To facilitate the securing of samples, the location of lots that are available for sampling can be identified by the producer or supplier so that suitable samples can be secured by the prospective purchaser or his representative.

1.5 — Testing

Admixtures are tested for acceptance for one or more of four reasons: (a) to determine compliance with a purchase specification; (b) to evaluate the effect of the admixture on the properties of the concrete to be made with job materials under the anticipated ambient conditions and construction procedures; (c) to determine within-lot uniformity of product; and (d) to provide data showing that any lot is the same as those previously supplied.

When required by the purchase contract or specifications, the admixture should be tested by a laboratory to show that it meets current ASTM or other applicable specifications before being allowed for use. The manufacturer of the admixture may be required to certify that individual lots meet the requirements of applicable standards.

1.5.1 — Quality control test methods are used by producers of admixtures to determine a product's compliance with the producer's own finished product specifications. Such test methods are often developed for the particular product and thus are not to be found in compilations of standard test methods. Standard test methods, as well as those recommended by individual producers, are available to evaluate admixtures from the consumer's viewpoint.

1.5.2 — Although qualifying tests, such as those of the ASTM, afford a valuable screening procedure for selection of admixture products, any substantial use in continuing production of concrete should be preceded by testing that allows observation and measurement of the performance of the product under concrete plant operating conditions in combination with the concrete-making materials then in use. Uniformity of results is usually as important or more important than the average result with respect to each significant property of the admixture or the concrete.

1.6 — Decision on use

Use of an admixture may increase or decrease the unit cost of the concrete. The cost of doing the work using an admixture should be compared with that of alternative materials or methods. The effect of a given admixture can sometimes be obtained, at least in some degree, by other means or by other admixtures. The cost of the ingredients making up a concrete mixture containing the admixture should be compared with the cost of ingredients of concrete without admixture.

1.6.1 — The cost of handling an extra ingredient and any effect the use of the admixture may have on the cost of transporting, placing, finishing, curing, and protecting the concrete should be taken into account. Frequently an admixture permits use of less expensive construction methods or even of less expensive structural design. For example, novel and economical designs of structural units have been predicated on use of retarding admixtures that permit placing of concrete over extended periods in homogeneous units of large size and great volume, thus reducing need for forming and the placing and joining of separate units. Use of air-entraining and water-reducing admixtures commonly makes possible the attaining of required physical properties of lightweight aggregate concrete at lower unit weight.

1.6.2 — Evaluation of the use of any given admixture or combination of admixtures should be based on the results obtained with the particular concrete in question under conditions expected on the job, since the results obtained are influenced to an important degree by the characteristics of the cement and aggregate and their relative proportions, construction practices, and ambient conditions. The plant operator, contractor, and owner of the construction project are interested in features other than the properties of the concrete that are measured by standard tests. Of primary concern may be workability, pumping qualities, placing and finishing characteristics, early strength development, reuse of forms and molds, appearance of formed surfaces, and so on. These features may dictate selection of an admixture and a dosage of the admixture more than do the properties that are covered by usual specifications.

CHAPTER 2 — PREPARATION AND BATCHING OF ADMIXTURES

2.1 — Introduction

The successful use of admixtures depends upon the use of an appropriate method of preparation and batching. A batching system requires accuracy, reproducibility, and speed. Failure in any one of these areas may significantly affect properties and performance of the concrete.

The preparation of any admixture prior to its introduction into the concrete mixture should follow the recommendations of the manufacturer and established and accepted procedures. Though not directly a part of the preparation process, attention should be directed toward proper storage of admixtures. Manufacturers' storage recommendations should be followed.

With respect to batching systems, admixtures may be grouped into three categories: (1) those materials introduced into the batch in liquid form; (2) finely divided mineral admixtures such as natural pozzolans, fly ashes, and silica flour; and (3) admixtures which are blended with inert materials serving as extenders to facilitate accurate weighing and distribution of the active ingredient in the concrete batch.

2.2 — Batching equipment

2.2.1 *General* — Powdered admixtures should be batched by weight, and paste or liquid admixtures by weight or volume. Most liquid admixture batchers for concrete in use at present are of the volumetric type.

Concrete batchers are rated by the Concrete Plant Standards[15] of the Concrete Plant Manufacturers' Bureau as being either manual, semiautomatic, or automatic. Most available admixture batching systems are adaptable to a manual or semiautomatic concrete batcher. An automatic concrete batcher requires much more complex admixture batching systems to enable complete interlocking to assure batching accuracy within specified tolerances, and to provide for the recording of the quantity batched if recorders are used. Most specifications require some means to allow visual inspection of quantity batched.

Several methods being employed to achieve control of amounts batched are described in the following sections.

2.2.2 *Positive displacement volumetric batching* — The following two types of admixture batchers are ideally suited for use on semiautomatic or fully automatic concrete batchers, since appropriate interlocks may easily be made for remote operation.

1. Meters: Positive displacement flow meters, similar to water meters, are used to meter admixture quantities. These may be equipped with electrical mechanisms that will emit pulses, for a given quantity of solution, to operate a preset electrical counter. However, most meters are calibrated for a given viscosity of liquid and care should be taken to maintain viscosity within the calibrated range or errors in metering will be introduced. Viscosity of liquid admixtures may vary widely with temperature.

2. Containers: Positive displacement measuring containers are equipped with floats. Linear movement of the float in a measuring unit of a known cross section represents the volume of the solution batched. These floats are usually geared to pulsing switches, potentiometers, or synchro transmitters which operate electrical preset counters or null devices respectively.

2.2.3 *Visual volumetric container* — The visual volumetric container is generally a manually operated system wherein the operator controls the filling and discharge of the calibrated visual container with manually operated valves. Containers of different size are available to meet the needs of plants of various capacities. Gravity feed systems are the least expensive and require the least maintenance. There are various hardware options available with this type of system, such as power inlet and outlet valves, reservoir pumps, and automatic float controls and air discharge mechanisms. Gravity flow from a typical calibrated container through a ¾ in. (2 cm) line is about 3 gal. per min (11 dm^3/min). Faster flow rates or discharge against gravity may be obtained with air pressure. Such containers should be designed for pressure operation. Usually, 0.5 psi (0.035 kgf/cm^2) of air pressure per foot of static head, plus 5 psi (0.35 kgf/cm^2) is sufficient.

2.2.4 *Timer controlled systems* — Although timer controlled systems may be used, their use is not recommended due to extreme inaccuracy and nonreproducibility. In essence, these systems involve the timing of flow through an orifice. Electrical timers can be affected by changes in power supply to the timer motor. Partial restriction of the orifice by foreign matter can seriously reduce accuracy. Changes in viscosity due to temperature change of the solution also cause variations in flow rate.

2.2.5 *Weigh batching* — Weigh batching is a feasible method of controlling the addition of admixtures. The type of scale, beam or dial, must be such that the required batching accuracy is obtained. An indicator should be included in a weigh system in order that the operator can be certain that all of the admixture weighed has been discharged. The indicator may be in the form of a sight glass if the weigh hopper is in the operator's view, or an electrical mechanism to signal when the hopper is empty, or when it has failed to discharge completely. A weigh batcher has the disadvantage that all liquid admixture dosages have to be converted from volume to weight. It is also often necessary to dilute admixture solutions to obtain sufficient quantity for accurate weighing.

2.3 — Batching tolerances

2.3.1 *Admixture batchers for manual and semiautomatic concrete batch plants* — Admixture batching systems for use in manual or semiautomatic plants should be capable of volumetric batching within an accuracy of ± 3 percent of the required volume, or one fluid ounce (30 cm^3), whichever is greater, unless otherwise stipulated in the job specifications. Weight batchers should be capable of weighing within an accuracy of ± 3 percent of the required weight. In no case should the quantity weighed be so small that 0.4 percent of the full scale capacity exceeds 3 percent of the required weight.

2.3.2 *Admixture batchers for automatic concrete batching systems with mixture selection* — An admixture batcher for use in an automatic concrete plant must be capable of being interlocked into the batching system as follows:

1. The charging valve of a volumetric batcher cannot be opened until the liquid level within the container has returned to zero within an accuracy of ± 0.3 percent of the capacity of the container, or 1 fluid ounce (30 cm^3), whichever is greater. Positive displacement flow meters equipped with pulsing devices are excluded from this requirement. Charging valves of weigh batchers cannot be opened until the scale has returned to zero balance within an accuracy of ± 0.3 percent of the capacity of the scale.

2. The charging valve cannot be opened if the discharge valve is open.

3. The discharge valve cannot be opened if the charging valve is open.

4. The discharge valve of a volumetric batcher cannot be opened until the designated volume is within an accuracy of ± 3 percent of the required volume or 1 fluid ounce (30 cm^3), whichever is greater, unless otherwise stipulated in the job specifications. The discharge valve of a weigh batcher cannot be opened until the designated weight of admixture in the batcher is within an accuracy of ± 3 percent of the required weight. In no case should the quantity weighed be so small that 0.4 percent of the full scale capacity exceeds 3 percent of the required weight.

2.4 — Liquid admixtures

2.4.1 *General* — Materials introduced into the concrete mixture as liquids generally fall into the following categories: air-entraining admixtures, water-reducing admixtures, water-reducing retarders, retarders, water-reducing accelerators, and accelerators. The dosage of these admixtures may vary from as little as 0.1 oz (3 cm^3) to as much as 64 oz (1900 cm^3) per 100 lb (45 kg) of cement.

2.4.1.1 Incompatibility of admixtures. Two or more admixtures may not be compatible in the same solution. It is therefore important that unless shown to be permissible by appropriate tests or the advice of the manufacturer, admixtures not be intermixed prior to their introduction into the concrete. It may also be desirable to introduce the admixtures into the mixer at separate times during mixing.

2.4.2 *Preparation*

2.4.2.1 From powders and particles. Some chemical admixtures are supplied as water-soluble solids requiring job mixing at the point of usage. Such job mixing may require that low concentration solutions be made due to difficulty in mixing. Under these conditions, the water in the solution must be considered a part of the total water content of the batch in order to maintain correct water-cement ratio. Manufacturers' recommendations should be followed carefully to assure complete solution of the product or to prepare a standard solution of uniform strength for easier use. Since certain admixtures may contain significant amounts of finely divided insoluble materials or active ingredients which may or may not be readily soluble or dispersible, it is important that precautions be taken to ensure that these constituents will be kept in a state of uniform suspension before actual batching. The practice of adding the water-soluble powders directly to the concrete mixture should usually be discouraged since the small quantity of material involved may not be adequately dispersed throughout the batch.

2.4.2.2 As ready-to-use liquid. Admixtures that are supplied as ready-to-use liquids may be of a much higher concentration than job-mixed solutions, and as a result any finely divided insoluble matter, if present, will tend to stay in suspension and continuous agitation may not be required.

2.4.3 *Batching*

2.4.3.1 General. The time of introduction and the rate of discharge of the admixture must be synchronized with other steps in the batching operation. Rate of discharge is critical. The entire amount of admixture should be added prior to the completion of the addition of the mixing water. The time at which certain chemical admixtures are introduced in the mixing cycle is also of major importance. For some cement-admixture combinations, varying the time at which they are added during mixing may result in varying degrees of retardation or acceleration, or significantly affect the water requirement of the mixture. Chemical admixtures in liquid form should not come in contact with dry cement. For any given condition or project, a procedure for controlling the time and rate of the admixture addition to the concrete batch should be established and adhered to closely. To assure uniform distribution of the admixture throughout the concrete mixture during the charging cycle, the rate of admixture discharge should be adjustable.

2.4.3.2 Protection and maintenance. It is important that batching systems receive periodic routine maintenance to prevent inaccuracies caused by such things as sticky valves, build up of foreign matter in meter bodies, and worn pumps. Whenever possible, batching system components should be located in areas that will provide protection from dust and from extreme temperatures, and be readily accessible for visual observation and maintenance. Storage tanks and mixing tanks should be inspected and cleaned periodically. This will prevent a build up of sediment or mold growth, or both, in the system.

2.4.3.3 Cold weather. Measures should be taken for cold weather protection of the solution and batching system, particularly with respect to protection of the solution from freezing in storage tanks and delivery lines. This may involve installation of immersion heaters in the tanks and wrapping lines with insulation or heating tapes, or both. Means should be provided to completely clear discharge lines to avoid retention of liquids susceptible to freezing. Usually an admixture that has frozen in storage tanks or drums may be reconstituted after thawing without detrimental effect on the quality of the admixture. However, it is advisable to obtain the manufacturer's recommendation regarding this point.

2.4.3.4 Vents. Bulk storage tanks, drums, and calibrated sight tubes should be vented so that they do not become air bound and restrict flow. Since vents tend to pull in atmospheric dust as tanks drain, they should include an air filter. Vents on sight tubes must extend above the liquid level of overhead gravity supply tanks.

2.4.3.5 Flushing. All systems should be supplied with a water inlet valve for flushing. Cool water should be passed through the system until it runs clear.

2.5 — Finely divided mineral admixtures

2.5.1 *Storage and handling* — Admixtures in this class, such as fly ash or natural pozzolan, are most readily conveyed and batched dry. It is necessary, therefore, to keep them dry during storage, conveying, and batching. Storage facilities should be as weathertight as for portland cement. Since the appearance (color and texture) of some finely divided mineral admixtures is close to that of portland cement, storage and auxiliary facilities for bulk material should be clearly marked to avoid errors in delivery and misuse. If compartmented bins are used for bulk mineral admixtures and cement, precautions should be taken in design, construction, and use, to avoid leakage from one compartment to another. Compartmented bins should be inspected frequently to make certain that no leaks develop. Bulk mineral admixtures may be conveyed to elevated storage pneumatically or by bucket elevators. If bucket elevators are used an adjustment in operating speed depending on the weight of the material may sometimes be needed. A lightweight material will not drop out of the buckets as quickly as cement, for instance; therefore, the buckets must move more slowly to allow time for the material to discharge into the chute.

2.5.2 *Conveying*

2.5.2.1 Gravity and air conveying. Most mineral admixtures flow by gravity quite readily when aerated. Low air pressures should be used for this purpose. As a result of this "flowability" these materials may be conveyed to the weigh hopper by gravity if storage is high enough to permit the connecting chute to have sufficient slope. If the slope is insufficient, an "air slide" or air chute is recommended. Manufacturers of such conveyors will recommend the best operating slope for different materials. A positive shutoff device at the outlet of the bin is necessary when conveying by these methods.

2.5.2.2 Screw conveying. If the conveying line from storage to weigh batcher is horizontal or at an elevating incline, a screw conveyor is recommended. For materials with a tendency to flow, such as fly ash, the feed of material may not be satisfactorily controlled merely by stopping and starting the screw. Some material may continue to flow around the flight channel of the screw after it has stopped. As a result, it may be necessary to install a shutoff at the outlet of the bin when using a screw with such admixtures. Another difficulty in conveying mineral admixtures by screw relates to their tendency to become quite dense and immobile when allowed to settle and lose air. If an unusually long screw conveyor is necessary, precautions may need to be taken to empty, or nearly empty, the screw before allowing it to be idle for many hours.

2.5.3 *Batching* — Batching of mineral admixtures should be by weight. Automatically controlled batching with appropriate interlocks to prevent duplication or omission is preferred and highly recommended. A "zero-empty" indicator could produce an electrical signal when the hopper has been discharged, or has failed to discharge completely. Materials in this class may be introduced satisfactorily with the cement and other components of the mix. They should not be charged into a wet mixer ahead of the other materials because of a tendency to stick to the sides of the mixer drum. Likewise, they should not be charged into the mixer along with the mixing water because of their tendency to ball up under such conditions. In production of lightweight aggregate concrete, superior results may sometimes be achieved by mixing aggregates, water, and mineral admixtures before adding the cement to the batch.

2.6 — Miscellaneous powdered admixtures

2.6.1 *General* — Examples of admixtures that may be in powdered form, other than pozzolanic materials, are certain gas-forming, coloring, grouting, expansion-producing, and retarding admixtures.

2.6.2 *Preparation* — When relatively small amounts of powdered admixtures are used, it is recommended they be blended with portland cement, or a vehicle material, such as pozzolan or pulverized stone to act as a carrying agent.

2.6.3 *Batching* — The blending procedure noted in Section 2.6.2 not only allows the active ingredient to be more easily distributed throughout the concrete mixture, but also facilitates accurate weighing. Since most of these materials are highly concentrated, it is important that every effort be made to control accurately the addition rate and provide facilities to preclude overdose or ommission of the admixtures as well as to insure the complete blending of the admixture with the vehicle material. Such operations may be expedited by batching of the admixture from packages that are of a size appropriate for a single batch, the packages being purchased as such or prepared beforehand by job personnel.

2.7 — Concluding statement

Additional information on preparation and batching of particular classes of admixtures will be found in the following chapters.

CHAPTER 3 — AIR-ENTRAINING ADMIXTURES

3.1 — Introduction

The ACI glossary of terms[16] defines an air-entraining agent as: "An addition for hydraulic cement or an admixture for concrete or mortar which causes air, usually in small quantity, to be incorporated in the form of minute bubbles [about 1 mm diameter or smaller*] in the concrete or mortar during mixing, usually to increase its workability and frost resistance." This report is concerned with those air-entraining agents which are added to the concrete batch immediately before or during its mixing, and are referred to as air-entraining admixtures.

3.2 — Effect

The primary purpose of air entrainment in concrete is to provide a high degree of resistance

*Represented as circular sections 1 mm in diameter or less on random surfaces cut through the concrete.

to the disruptive action of freezing and thawing and of deicing chemicals. However, the use of entrained air is recommended in concrete for other reasons, also. Air entrainment favorably alters a number of the properties of freshly-mixed concrete. Plasticity and workability are improved, enabling a reduction in water content. Uniformity of placement and of consolidation can be achieved more readily, thus reducing segregation. Bleeding also is reduced. In addition to improving the resistance of hardened concrete to freezing and thawing and deicer scaling, air entrainment also increases the resistance of concrete to sulfate action. Watertightness of air-entrained concrete is superior to that of concrete without air entrainment. Unit weight is reduced. Air entrainment, while improving both workability and durability, may reduce strength. Within the range of air contents normally used, the decrease in strength usually is about proportional to the amount of air entrained. For most types of exposed concrete a slight reduction in strength is far less significant than the improved resistance to frost action. The reduction in strength will rarely exceed 15 percent in the case of compressive strength and 10 percent in the case of flexural strength. These figures are for equal cement content and with the sand and water content of the air-entrained concrete reduced to the extent permitted by the increased workability of this type of mixture.

3.3 — Applications

As stated in Section 3.2, the use of entrained air is recommended in concrete for several reasons. Because of its greatly improved resistance to frost action, air-entrained concrete should be used wherever concrete is exposed to freezing and thawing, to the action of salts used for deicing, or to other potentially damaging environments. Its use is also desirable wherever there is a need for watertightness. Air entrainment improves the workability of concrete. It is particularly effective in lean mixtures, which otherwise may be harsh and difficult to work. It is common practice to provide air entrainment in various kinds of lightweight aggregate concrete, including not only insulating and fill concrete, but also in structural lightweight concrete. Admixtures for cellular concrete are not included in this guide since the subject is covered by ACI Committee 523.

There is no general agreement on the benefits resulting from the use of an air-entraining admixture in the manufacture of concrete block.[17-19] However, satisfactory results using air-entraining admixtures have been reported in the manufacture of cast stone and concrete pipe.

3.4 — Evaluation and selection

3.4.1 — Many materials are capable of functioning as air-entraining admixtures; these include: (1) salts of wood resins, (2) some synthetic detergents, (3) salts of sulfonated lignin, (4) salts of petroleum acids, (5) salts of proteinaceous materials, (6) fatty and resinous acids and their salts, and (7) organic salts of sulfonated hydrocarbons. Some materials, such as hydrogen peroxide and powdered aluminum metal, can be used to entrain gas bubbles in cementitious mixtures but are not considered as acceptable air-entraining admixtures, since they do not necessarily produce an air-void system which will enhance resistance to freezing and thawing.

3.4.2 — To achieve the desired improvement in frost resistance, intentionally entrained air must have certain characteristics. Not only is the total volume of air of importance, but more importantly the size and distribution of the air voids must be such as to provide efficient protection to the cement paste. The air-void system must be characterized by a large number of small voids, uniformly distributed throughout the cement paste.

3.4.3 — To assure that an air-entraining admixture produces a desirable air-void system, it should meet the requirements of ASTM C 260.[7] This specification sets limits on the effects which any given air-entraining admixture under test may exert on bleeding, time of setting, compressive and flexural strength, resistance to freezing and thawing, and length change on drying of a hardened concrete mixture, in comparison with a similar concrete mixture containing a standard reference air-entraining admixture, such as neutralized vinsol resin. The methods by which these effects may be determined are given in ASTM C 233.[20] Extensive testing and experience have shown that concretes having total air contents in the range of the recommended air contents shown in ACI 211.1-70[21] will have the proper size and distribution of air voids when the air-entraining admixture used meets the requirements of ASTM C 260.[7]

3.5 — Control of purchase

Most of the commercial air-entraining admixtures available, sold under various trade names, are in liquid form, although a few are powders, flakes, or semisolids. The proprietary name and the net quantity in pounds (kilograms) or gallons (cubic decimeters) should be plainly indicated on the package or containers in which the admixture is delivered. The admixture should be uniform within each batch and uniform between batches and between shipments. Acceptance testing should be as stated in Section 1.5.

3.6 — Batching, use, and storage

To achieve the greatest uniformity in a concrete mixture and in successive batches, it is recommended that air-entraining admixtures be added to the mixture in the form of solutions rather than solids.

Generally, only small quantities of air-entraining admixtures are required to entrain the desired amount of air. These are of the order of 0.05 percent of active ingredient by weight of the cement. If the admixture is in the form of powder, flakes or semisolids, a proper solution must be prepared prior to use, following the recommendaions of the manufacturer.

If the manufacturer's recommended amounts of an air-entraining admixture do not result in the desired air content, it is necessary to adjust the amount of admixture added. For any given set of conditions and materials, the amount of air entrained is roughly proportional to the quantity of agent used. However, in some cases a "ceiling" may be reached and it may be necessary to change the type of air-entraining admixture or to achieve the desired result some other way (e.g., change the cement).

Attention should be given to proper storage of air-entraining admixtures. The manufacturer's storage recommendations should be followed. If they are not available, they should be requested from the manufacturer. Air-entraining admixtures are not usually damaged by freezing, but the manufacturer's instructions should be followed regarding the effects of freezing on his product. After completion of tests, an admixture stored at the point of manufacture for more than 6 months prior to shipment, or an admixture in local storage in the hands of a vendor or contractor for more than 6 months, should be retested before use and rejected if it fails to conform to any of the applicable requirements of ASTM C 260.[7] Further information is given in Chapter 2.

3.7 — Proportioning of concrete

The proportioning of air-entrained concrete is similar to that of non-air-entrained concrete. It is recommended that methods of proportioning air-entrained concrete follow the procedures of ACI Committee 211.[21] This procedure incorporates the reduction in water and sand permitted by the improved workability of air-entrained concrete.

Air entrainment may reduce strength. However, when cement content and workability are maintained constant, the reduction in strength is partially or entirely offset by the resulting reduction in water-cement ratio. This is particularly true of lean concretes or those containing a small maximum size aggregate. Such concretes therefore may not have their strengths reduced; they may even be increased by the use of air entrainment.

3.8 — Factors influencing amount of entrained air

There are numerous factors that can influence the amount of air entrained in concrete. The amount of air-entraining admixture required to obtain a given air content will vary widely depending on the particle shape and grading of the aggregates used. For example, concrete using crushed fine aggregate may require up to twice as much admixture as is needed when rounded natural sand is used. Organic impurities in the aggregate may either increase or decrease the air-entraining admixture requirements depending on the nature of the impurity. An increase of the hardness of water will generally decrease the effectiveness of the air-entraining admixture.

As the cement content increases, the air-entraining potential of a given amount of an admixture will tend to diminish; an increase in cement fineness will also result in a decrease in the amount of air entrained. High-alkali cements generally require smaller amounts of air-entraining admixtures to obtain a particular air content than do low-alkali cements. Larger amounts of air-entraining admixture are generally required in concrete containing high-early-strength (Type III)[22] portland cement and portland-pozzolan cement (Type IP).[23]

Increasing the amount of finely divided materials in concrete by the use of fly ash or other pozzolans, carbon black, or bentonite usually decreases the amount of air entrained for a given amount of admixture. A given amount of an air-entraining admixture generally produces slightly more air where calcium chloride is used as an accelerator. Similarly, the amount of air-entraining admixture required to produce a given air content of concrete may be reduced one-third or more when used with a water-reducing admixture. Various types of admixtures can influence the air content and quality of the air-void system, and therefore, special care should be taken when such admixtures are used in conjunction with air-entraining admixtures to assure that there is compatibility.

Air content generally increases with slump. Relatively wet mixtures may, however, have a spacing factor that is larger, and therefore less desirable, than drier mixes. An increase in water-cement ratio is likely to result in an increase in air content. Less air is entrained as the temperature of the concrete increases.

The amount of air entrained varies with the type and condition of the mixer, the amount of concrete being mixed, and the rate of mixing. The amount of air entrained by any given mixer will decrease appreciably as the blades become worn, or as the mixing action is impaired if hardened mortar is allowed to accumulate in the drums and on the blades. If there is a significant departure from the rated capacity of the mixer, there may be decreases or increases in air content. Adams and Kennedy[24] found for various mixers and mixtures, in the laboratory, that air content increased from a level of about 4 percent to from 4 to 8 percent as the batch size was increased from slightly under 40 percent to slightly over 100 percent of rated mixer capacity. The amount of entrained air increases with mixing time up to a point, beyond which it slowly decreases. However, the air-void system, as characterized by specific surface and spacing factor, is generally not harmed by prolonged agitation. If more water is added to develop the desired slump, the air content should be checked since some adjustment may be required; addition of water without thorough or complete mixing may result in nonuniform distribution of air and water within the batch. See the ACI 304 report for further details.[25]

The type and degree of consolidation used in placing concrete can reduce the air content. Fortunately, indications are that the air loss as a result of these manipulations consists of the larger bubbles of entrapped air which contribute little if anything toward the beneficial influences of entrained air.

3.9 — Control of concrete

To achieve the benefits of entrained air in a consistent manner requires relatively close control of the air content of the concrete. Tests for air content of freshly mixed concrete should be made at regular intervals for routine control purposes during placing. Tests should also be made at any time there is reason to suspect a change in air content.

The significant air content is that present in concrete in place after consolidation. Losses of air which occur due to handling and transportation and to consolidation may not be reflected by tests for air content of concrete taken at the mixer (see report of ACI Committee 309). Because of this, air content often should be checked at the point of discharge into the forms.

There are three standard ASTM methods for measuring the air content of fresh concrete: (1) the gravimetric method ASTM C 138;[26] (2) the pressure method, ASTM C 231,[27] and (3) the volumetric method, ASTM C 173.[28] The most widely used is the pressure method, which however may not be applicable to lightweight concretes. The volumetric method is applicable to lightweight concretes. An adaptation of the volumetric method using the so-called Chace meter, in which a small sample of mortar from the concrete is used, is rapid and convenient, applicable to all concretes, and useful to determine abrupt changes in air content which may necessitate more accurate measurements.

The above mentioned methods for determining air content of freshly mixed concrete measure only air volume and not the air-void characteristics. The air content, spacing factor, and other significant parameters of the air-void system in hardened concrete can only be determined microscopically by methods such as the linear traverse modified point count procedures as described in ASTM C 457.[29] Use of these methods in coordination with investigations of proportioning of concrete for new projects provides greater assurance that concrete of satisfactory resistance to freezing and thawing will be obtained. It has been shown, however, that the air content of a concrete mixture is generally indicative of the adequacy of the air-void system when the air-entraining admixture used meets ASTM C 260.[7]

The properties of the concrete-making materials, the proportioning of the concrete mixture, and all aspects of mixing, handling, and placing should be maintained as constant as possible in order that the air content will be uniform and within the range specified for the work. This is important primarily because too much air detracts unnecessarily from strength without a commensurate improvement in durability, whereas too little will fail to provide desired workability and durability.

Proper inspection should insure that air-entraining admixtures conform to the appropriate specifications so far as can be determined in the field; that they are stored without contamination or deterioration; that they are accurately batched and that they are introduced into the concrete mixture as specified. Trouble has frequently occurred from failure to batch the admixture, either through operator error or equipment malfunction. See Chapter 2 for further details. The air content of the concrete should be checked and controlled during the course of the work in accordance with the recommendations of ACI Committee 311 as reported in the *ACI Manual of Concrete Inspection*.[30] Practices causing excessive air loss should be corrected or compensating additional air should be entrained initially.

CHAPTER 4 — ACCELERATING ADMIXTURES

4.1 — Introduction

An accelerating admixture is a material added to the concrete for the purpose of shortening the setting time and accelerating early strength development of concrete.

By far the best known and most widely used accelerator is calcium chloride. Many other materials have been found to accelerate the strength gain of concrete but, in general, they are seldom used, and only limited information concerning their effect on the properties of concrete is available. Most of the information given on accelerators applies mainly to the use of calcium chloride. Other chemicals which accelerate the rate of hardening of concrete include some other soluble chlorides, soluble carbonates, silicates, fluosilicates, alkali hydroxides, and some organic compounds such as triethanolamine.

4.2 — Consideration of use

Accelerating admixtures are useful for modifying the properties of portland cement concrete, particularly in cold weather, to: (a) expedite the start of finishing operations and, when necessary, the application of insulation for protection; (b) reduce the time required for proper curing and protection; (c) increase the rate of early strength development so as to permit earlier removal of forms and earlier opening of the construction for service; and (d) permit more efficient plugging of leaks against hydraulic pressure. The use of accelerators in cold weather concrete is usually not sufficient in itself to counteract effects of low temperature. Recommendations for cold weather concreting usually include such things as heating the ingredients, providing insulation and application of external heat (see ACI 306 recommendations[31]).

Accelerators should be used with care in hot weather. Some of the detrimental effects which may result are very high rate of evolution of heat of hydration, rapid setting, and shrinkage cracks.

Accelerators should never be used as antifreeze agents for concrete. In the quantities normally used, the freezing point of the concrete is lowered only a negligible amount, less than 2 deg C. No materials are known which will substantially lower the freezing point of the water in concrete without being harmful to the concrete in other respects.

4.3 — Effect on freshly mixed and hardened concrete

The effects of accelerators on some of the properties of concrete are as follows:

4.3.1 *Setting time* — The setting time, initial and final, is reduced. The amount of reduction varies with the amount of accelerator used, the temperature of the concrete, and the ambient temperature. Excessive amounts of the accelerator may cause rapid setting.

4.3.2 *Air entrainment* — Less air-entraining admixture is required to produce the required air content. However, in some cases larger bubble sizes and higher spacing factors are obtained.

4.3.3 *Heat of hydration* — Earlier heat release is obtained but there is no appreciable effect on the total heat of hydration.

4.3.4 *Strength* — Compressive strength is increased substantially at early ages. The ultimate strength may be reduced slightly. The increase in flexural strength is usually less than that of the compressive strength.

4.3.5 *Volume change* — It is generally considered that the volume change is increased for both moist curing and under drying conditions. There is a question of the degree of the effect caused by the accelerators as opposed to other factors influencing volume change.

4.3.6 *Durability* — The resistance to freezing and thawing and to scaling caused by the use of deicing salts is increased at early ages, but may be decreased at later ages.

4.3.7 *Sulfate resistance* — The resistance to sulfate attack is decreased.

4.3.8 *Alkali-aggregate reaction* — The expansion produced by alkali-aggregate reaction is greater. This can easily be controlled by the use of low alkali cement or pozzolans.

4.3.9 *Corrosion of metals* — Calcium chloride should not be used when steam curing is employed unless tests of the specific application demonstrate the absence of objectionable corrosion. Severe corrosion of galvanized steel sheet permanent forms have been attributed to the use of calcium chloride. The use of calcium chloride in recommended amounts does not cause progressive corrosion of conventional steel reinforcement in typical reinforced concrete under normal conditions where the bars have sufficient concrete cover.

Stannous chloride when properly used acts as an accelerator and does not cause corrosion of the steel even when steam curing is used.

Woods[32] provides information on corrosion phenomena involving miscellaneous embedded metals. Sections 3.4.1 and 3.6.1 of ACI 318-71[33] contain the following requirements, respectively:

a. "In addition, the mixing water for prestressed concrete or for concrete which will contain aluminum embedments, including that portion of the mixing water contributed in the form of free moisture on the aggregates, shall not contain deleterious amounts of chloride ion."

b. "Admixtures containing chloride ions shall not be used in prestressed concrete or in concrete containing aluminum embedments if their use will produce a deleterious concentration of chloride ion in the mixing water."

Although there is no definite limit given by ACI 318, the Commentary [34] expresses the opinion that chloride ion content greater than 400 or 500 ppm might be considered dangerous and states that ACI Committee 222 recommends levels well below these values.

ACI Committee 301[35] is more restrictive. The proposed revision of ACI 301-66 states: "For prestressed concrete and for all concrete in which aluminum or galvanized metal is to be embedded, it shall be demonstrated by test that the mixing water of the concrete, including that contributed by the aggregates, and any admixture used, will not contain more than 150 ppm of chloride ion." ACI 318 prohibits use of calcium chloride in grout used in prestressed concrete; 301 prohibits all admixtures containing chlorides, fluorides, and nitrates for use in such grout.

4.4 — Evaluation and selection

The decision whether or not an accelerator should be used in concrete is often a matter of economics. Frequently, the same results may be obtained by other means, such as (a) the use of a Type III cement,[22] (b) the use of additional cement, (c) the use of longer or different method of curing and protection, or (d) a combination of these. In most cases, the use of an accelerator is the most economical and convenient method of obtaining the desired results.

4.5 — Control of purchase

Calcium chloride, which is available in two forms, should meet requirements of ASTM D 98.[6] Regular flake form should have a minimum of 77 percent $CaCl_2$ and should meet the requirements for Type I. Concentrated flake, pellet, or granular form should have a minimum of 94 percent $CaCl_2$ and should meet the ASTM requirements for Type II. For other materials, the formula should be that specified by the producer. All accelerators should meet the requirements of ASTM C 494[5] for Type C or E.

4.6 — Batching and use

The exact amount of accelerator needed to obtain the desired acceleration of the setting time and strength development depends on local conditions but generally 1 to 2 percent of the weight of cement is added.

Calcium chloride should be introduced into the concrete mixture in solution form. Preparation of a standard solution from dry calcium chloride requires that the user be aware of the two different concentrations of calcium chloride described above. For example, two 100-lb (45-kg) bags of Type I calcium chloride or two 80-lb (36-kg) bags of Type II calcium chloride can be used to prepare 50 gal. (190 dm^3) of standard solution. In this solution each quart (0.9 dm^3) contains the equivalent of 1 lb (0.45 kg) of regular flake calcium chloride. This provides a convenient method to add approximately 1 or 2 percent regular flake calcium chloride for each bag of cement used by adding 1 or 2 quarts (0.9 or 1.9 dm^3), respectively, of standard solution. Since regular flake calcium chloride contains only 77 percent $CaCl_2$, one quart of the standard solution will provide about ¾ lb (0.3 kg) pure $CaCl_2$, or 0.8 percent of the weight of a 94-lb (43-kg) bag of cement.

When preparing a standard solution from dry material, the calcium chloride should be added to the water rather than the water to the calcium chloride, as a coating may form which is difficult to dissolve. The concentration of the solution may be verified by checking the specific gravity, which should be 1.28 ± 0.05.

In some locations calcium chloride liquor is available. Since there is no ASTM specification for calcium chloride liquor, the user will have to rely on the manufacturer's statement as to the quality.

For either the standard solution or liquor, the amount added shall be deducted from the amount of water required for the desired water-cement ratio. Automatic batching systems are available and are recommended to insure accurate and uniform addition of calcium chloride in liquid form.

4.7 — Proportions of concrete

The mixture proportions for concrete containing an accelerator should be the same as those without the accelerator. Usually the recommended maximum dosage of calcium chloride is 2 percent by weight of portland cement.

4.8 — Control of concrete

The usual tests should be made for the control of concrete, such as slump, unit weight, and air content. If difficulty is encountered in proper consolidation or finishing of the concrete, the amount of accelerator used should be checked.

CHAPTER 5 — WATER-REDUCING AND SET-CONTROLLING ADMIXTURES

5.1 — Definition

Water-reducing and set-controlling admixtures are water-soluble organic or combined organic and inorganic materials which reduce the water requirement of concrete for a given consistency or which modify the rate of setting or hardening of the concrete or both. Accelerators are discussed separately in Chapter 4. Conventional air-entraining admixtures, discussed in Chapter 3, are not considered as water-reducing admixtures although one effect of air entrainment is to reduce water.

5.2 — Types

ASTM C 494[5] classifies chemical admixtures into the following types:

Type A Water-reducing
Type B Retarding
Type C Accelerating (See Chapter 4)
Type D Water-reducing and retarding
Type E Water-reducing and accelerating

The user may specify the type of water-reducing and set-controlling admixture in accordance with this ASTM specification or specify other requirements. Types A, B, D, and E are discussed in this chapter.

5.3 — Materials

The following classes of materials are used as water-reducing or set-controlling admixtures or both:

1. Salts of lignosulfonic acids (lignosulfonates)
2. Modifications and derivatives or formulations of lignosulfonic acids and their salts
3. Salts of hydroxylated carboxylic acids
4. Modifications and derivatives or formulations of hydroxylated carboxylic acids and their salts
5. Other materials, including carbohydrates, zinc salts, borates, phosphates, chlorides, amines and their derivatives, and various hydroxylated polymers, such as polysaccharides, certain cellulose ethers, certain melamine derivatives, and certain silicones.

5.4 — Effects on properties of concrete

5.4.1 *General* — The effects on the properties of concrete of the five types of admixtures given in Section 5.2 may vary considerably depending on the type and depending on the materials used in their formulation given in Section 5.3. They may also vary with the properties of the portland cement and other materials with which they are used. The effects noted below are a general guide. More detailed and specific information may be obtained from the producers of proprietary admixtures, from information gained from their actual use, and from laboratory studies of their effects under conditions simulating their usage.

5.4.2 *Effects on fresh concrete*

5.4.2.1 Water reduction. At the same slump and air content, the water-reducing Type A, D, and E admixtures may reduce the water requirement of concrete up to 10 percent. ASTM C 494[5] requires a water reduction of at least 5 percent for these types. These water reductions are for dosages of the admixture considered as normal without having adverse effects on other properties of concrete. The water reduction will vary with the admixture, the dosage, the properties of the cement and the concrete mixture.

5.4.2.2 Rate of setting or hardening. The rate of setting or of hardening of concrete may be modified so that it is either retarded or accelerated to varying degrees, depending on the material or materials and relative amounts used, properties of the cement, and other conditions of usage. Type A admixture is required (ASTM C 494) to give initial and final setting times in concrete not more than 1 hr earlier nor more than 1½ hr later than the reference concrete without admixtures. Types B and D admixtures are required to retard the concrete set from 1 to 3½ hr, compared with the reference concrete. Types C and E are required to accelerate the concrete between 1 and 3½ hr.

The setting times may vary with ambient temperature conditions,* properties of the cement, type of admixture, dosage of admixture, and the concrete mixture.

5.4.2.3 Air entrainment. Lignosulfonates (salts of lignosulfonic acids) usually produce some air entrainment, the amount varying with the specific admixture, the dosage, and other factors which are known to produce varying amounts of air in concrete. Salts of hydroxylated carboxylic acid admixtures normally do not entrain air. Water-reducing admixtures may have a supplemental air-entraining admixture included in their formulation to produce air-entrained concrete. With formulations which contain an air-entraining admixture there may be some loss of control of the air content of the concrete if the rate of use of the admixture is adjusted to accommodate requirements for setting time or strength development of the concrete. Therefore, when entrained

*Lower temperatures give longer setting times, less acceleration, and greater retardation—higher temperatures give shorter setting times, more acceleration, and less retardation.

air is specified, the air-entraining admixture should be added to the concrete mixture separately. Air-entraining admixtures may be used separately with all types of water-reducing and set-controlling admixtures. Air entrainment produced by the water-reducing admixture should have an acceptable effect on the concrete with respect to air-void characteristics as discussed in Chapter 3. When lignosulfonates are used and air entrainment is specified, the amount of air-entraining admixture must be decreased. With salts of hydroxylated carboxylic acids, the amount of air-entraining admixture must also be reduced for a given air content, although these materials do not normally entrain air when used alone. When air entrainment is not desired or if the amount of entrained air is excessive with admixtures containing lignosulfonates or other air producing materials, the air content of concrete may be reduced by the use of an air-detraining admixture such as tri-n-butyl phosphate, although any gross adjustment by this means may impair freezing-and-thawing resistance of the concrete by enlargement of the air voids, thereby producing an undesirably high value of spacing factor at a given air content.[36]

5.4.2.4 Bleeding. Salts of hydroxylated carboxylic acids cause an increase in the bleeding of concrete. Lignosulfonates usually do not cause an increase in bleeding, and commonly decrease bleeding. Other classes of materials vary in their effect on bleeding.

5.4.2.5 Workability. Types A, D, and E admixtures increase the slump of the concrete, if the water content of the mixture is held constant. At the same slump, concrete containing water-reducing admixtures usually has improved workability as judged by its placeability, less segregation, and improved response to vibration. When water-reducing admixtures are used, a given change in water content produces a greater change in slump than for comparable concrete without the admixture. The addition of a water-reducing admixture to mixed concrete at the job site to attempt to restore lost workability (slump) is not recommended because of the absence of facilities for accurate dosage and the improbability of achieving adequately uniform distribution of the admixture through the batch.

5.4.2.6 Heat of hydration and temperature rise. Adiabatic temperature rise and heat of hydration of concrete are not reduced at the same cement content when these classes of admixtures are used. If the cement content of concrete is reduced, the heat liberated per unit volume of concrete and the temperature rise are reduced due to the reduction in cement content. The time at which the major early heat generated by the hydration of the cement occurs may be changed (with retardation it occurs later; with acceleration it occurs earlier). This may modify slightly the temperature rise of the concrete under job conditions.

5.4.2.7 Slump loss. Tests show that the slump loss of concrete containing these admixtures is usually slightly greater than for comparable concrete without admixture. However, with equal water content, the higher slump obtained by the use of these admixtures may allow a longer period between mixing and placing.

5.4.3 *Effects on hardened concrete*

5.4.3.1 Strength. Types A and E admixtures give increased concrete strength at all ages. Concretes having equal cement contents, slump, and air contents, which contain Types B and D admixtures and meet the setting time requirements of ASTM C 494, will generally have compressive strengths at least equal to that of comparable concrete without the admixture at early ages of about 16 or 18 to 48 hr. At 28 days, the compressive strength may be increased from 15 to 25 percent. At later ages the percentage strength increase is generally less. Type E admixture is formulated to be accelerating; therefore the strength increase occurs earlier. The strength increase with the water-reducing admixtures is greater than that expected from the reduction in water-cement ratio; or, at the same water-cement ratio, and cement content, concrete with the admixture is stronger than concrete without the admixture. Flexural strength is increased less than compressive strength.

5.4.3.2 Shrinkage. There is much conflicting information on the effect of these admixtures on the shrinkage of concrete. Some may increase or decrease shrinkage, depending on their chemical composition and the characteristics of concreting materials. Usually the difference is not great and sometimes less than the testing errors. The method of test has a bearing on the results.

5.4.3.3 Durability. The resistance of concrete to freezing and thawing and to scaling is primarily related to the characteristics of the air-void system as discussed in Chapter 3. Some improvement in resistance to freezing and thawing beyond that due to entrained air results from water reduction and increased strength. A small increase in resistance to effects of aggressive water or aggressive soils results from water reduction, decreased permeability, and increased strength.

5.4.3.4 Other properties. In general, modulus of elasticity and bond to reinforcing steel are improved and creep is decreased by these admixtures. This is considered to be due to water reduction and an increase in strength. An increase in abrasion resistance and a decrease in permeability are also related to decrease in water content and an increase in strength. Some admixtures may contain chlorides such as calcium chloride. Consideration should be given to the potential corrosive effect of such admixtures on embedded materials, particularly prestressed steel or aluminum (see Section 4.3.9), and to the lower resistance of concrete containing chlorides to the effects of sulfate soils or water.

5.5 — Uses of water-reducing and set-controlling admixtures

5.5.1 *Water reduction* — The following applications of water reduction are important:

1. Economical proportioning of the concrete mixture, including the use of lower cement contents for a given strength and lessening of problems associated with aggregates that due to poor gradation or other reasons cause high water requirement
2. Reduction of temperature rise in larger or massive concrete sections because of lower cement content
3. Meeting requirements of job specifications such as maximum permissible water-cement ratio and early development of strength and modulus of elasticity, as in the production of prestressed concrete
4. Improvement of the quality of fresh concrete as a result of improved workability, reduced water content for a given consistency, or increased slump at constant or reduced water content. This is particularly desirable for concrete which is to be placed in heavily reinforced sections, under water, or by pumping. Increased rate of slump loss may reduce this advantage

5.5.2 *Retardation* — The following applications of retardation of setting are important:

1. Compensation for adverse ambient temperature conditions particularly in hot weather. Extensive use is made of retarding admixtures to permit proper placement and finishing and to overcome damaging and accelerating effects of high temperatures
2. Control of setting of large structural units to keep concrete workable throughout the entire placing period. This is particularly important for the elimination of cold joints and discontinuities in large structural units. Also control of setting may prevent cracking of concrete beams, bridge decks, and composite construction due to form deflection or movement associated with placing of adjacent units. Adjustment of the dosage as placement proceeds can permit various portions of a unit, a large post-tensioned beam for example, to attain a given level of early strength at approximately the same time

5.5.3 *Acceleration* — Applications where the acceleration of setting is important are discussed in Chapter 4.

5.6 — Proportioning concrete mixtures

Concrete mixtures containing admixtures which reduce water demand, entrain air, or otherwise cause a change in the yield of the mixture need to be proportioned to take these factors into account. Procedures outlined in ACI 211.1-70[21] and ACI 211.2-69[37] should be followed. In general, changes in water requirement, air content, or cement content may be compensated for by appropriate changes in fine aggregate content, keeping the proportion of mortar to coarse aggregate constant.

5.7 — Factors affecting performance

The specific effect of water-reducing and set-controlling-retarding admixtures varies with the composition of cement, water-cement ratio, temperature of the concrete, ambient temperatures, type of admixture, amount of admixture used, and other factors or job conditions.

Different sources and types of cement or different lots of cement from the same source, because of variations in chemical composition, or fineness, or both, may require different amounts of the admixture to obtain the desired results. The effectiveness of the admixture seems to be related primarily to the amount of tricalcium aluminate (C_3A) and the alkali (Na_2O and K_2O) content.[38] The sulfur trioxide (SO_3) content also may have a marked influence on the effect of the admixture on the time of setting of the concrete.[39-41] Low SO_3 may produce greater or excessive retardation. Early stiffening has been observed in a few cases.

In general, the quantity of water-reducing admixture required to produce the desired results will vary less with changes in cement composition or other mixture conditions than is true for the set-controlling admixtures. These latter types are intended to delay or accelerate the set of the concrete for a predetermined period at a given temperature. Slight changes from this temperature do not require a change in addition rate, but if either the temperature of the concrete or the ambient temperature varies more than 10 F (6 C) from that anticipated, a change in addition rate is generally desirable to maintain the desired retardation. The higher the temperature, the more admixture that will be required to produce a given degree of retardation and the less required for a given degree of acceleration.

The effectiveness of the water-reducing admixtures varies with the water-cement ratio of the mixture. Data show that increased slump can be achieved with less increase in water content when a water-reducing admixture is used than would be required otherwise.[42]

The addition of these materials to the mixture in liquid form is highly desirable to obtain a more uniform distribution throughout the concrete mass within the time allotted to adequately and properly mix the concrete. Care should also be taken when using liquid admixtures to avoid adding them directly to the cement or to dry, absorptive aggregate. A fixed procedure for the method and time of batching the admixture should be followed for each job. See Chapter 2 for procedures to be followed in mixing and batching admixtures.

The time of addition of a retarding admixture has a marked effect on the results obtained.[43,44] A delay of ½ to 2 min in adding the admixture after all other materials are batched and mixing has started will often result in greater increase in slump and retardation than normally expected.

5.8 — Testing methods and specifications

The testing of water-reducing and set-controlling admixtures should be conducted on concretes prepared both with and without the admixture to be evaluated. The admixture should be added in the manner recommended by the manufacturer and in the amount necessary to comply with the appropriate requirements of the job specifications. The tests should be made in accordance with ASTM C 494[5] and the admixtures should meet the requirements of ASTM C 494. In this evaluation, the concrete mixtures being compared are required to have slumps of 2½ ± ½ in. (63 ± 12 mm), and their air contents must not differ more than 0.5 percent.

When an admixture is being evaluated for a given job the materials to be used in that job should be used in the trial mixture program. The trial mixture program should be designed to determine the effect of the admixture on the properties of the concrete under the conditions of use, particularly with respect to temperature.

The time of setting should be determined in strict accordance with ASTM C 403[45] in order to insure reproducibility of results.

The specification limits given in ASTM C 494 take into account the variation of test data, which is greater when comparing concretes than when no comparison is required, as is the case of usual specifications.[46] The lower levels required for properties of the concretes with admixtures are designed to insure equal performance between the concretes under test, recognizing the inescapable statistical variation.

5.9 — Storage of admixtures

Powdered admixtures generally have an indefinite shelf life if stored dry and at suitable temperatures. Liquid admixtures may freeze or precipitate at low temperatures. Freezing may permanently damage some liquid admixtures. Other liquid admixtures may be frozen and thawed without damage. The manufacturer's storage directions should be followed.

5.10 — Quality control

The most practical means of insuring quality would be based upon index tests which, although not specific or definitive, can be used to control uniformity of the product. Suggested tests for this purpose are as follows:

1. Observation of physical nature
2. Determination of moisture content of solid products
3. Determination of pH of standard solutions
4. Determination of specific gravity or solids content of liquid admixtures
5. Analysis for specific ingredients such as percentage chlorides, carbohydrates or other compounds of a special interest
6. Infrared or ultraviolet spectroscopy to identify active constituents

CHAPTER 6 — FINELY DIVIDED MINERAL ADMIXTURES

6.1 — Types of finely divided material

6.1.1 *Relatively chemically inert materials* — This class includes such materials as ground quartz, ground limestone, bentonite, hydrated lime, and talc.

6.1.2 *Cementitious materials* — The cementitious materials include natural cements, hydraulic limes, slag cements (mixtures of blast-furnace slag and lime), and granulated iron blast-furnance slag.

6.1.3 *Pozzolans* — Pozzolan is defined in ASTM C 219[47] as "a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties." Fly ash, volcanic glass, diatomaceous earths, and some shales or clays, either heat-treated or raw, are examples of pozzolanic materials.

6.2 — Effect on properties of the concrete

6.2.1 *Freshly mixed concrete* — Where the available concrete aggregates are deficient in the finer particle sizes, particularly material passing the No. 200 sieve, the use of a finely divided mineral admixture can reduce bleeding and segregation and increase the strength of concrete by supplying those fines missing from the aggregate.[3] When an appropriate proportion of a finely divided mineral admixture is used, no increase in water content of the concrete is required. A favorable particle shape and a satisfactory fineness of the pozzolan are necessary qualities if a low water content is to be achieved. For example, coarse pozzolan of poor particle shape, such as volcanic glasses, may require an increase in water content of the concrete for given slump, and may thereby contribute to excessive bleeding and segregation of the fresh concrete.[48]

Addition of a finely divided mineral admixture to concrete mixtures not deficient in fines, particularly mixtures rich in portland cement, usually decreases workability for given water content. For that reason, the addition of finely divided mineral admixtures to such concrete without a reduction in cement content often entails an increase in the total water content of the concrete, and may result in an increase in drying shrinkage and absorptivity and a decrease in strength. Finely divided mineral admixtures should not be added to an existing concrete mixture without accomodating the characteristics of the admixture itself and its effects.[49] Fineness, water requirement, specific gravity, effect on air entrainment, and concrete yield are some of the factors to be considered. In general, the higher the specific surface of the admixture, the smaller the proportion required to produce a given effect on workability. Proportioning concrete with a finely divided mineral admixture is discussed below.

Finely divided mineral admixtures having a specific gravity less than that of normal-weight fine aggregate are sometimes used in lieu of such fine aggregate in lightweight concrete mixtures to reduce unit weight while maintaining workability.

Concrete is sometimes proportioned with finely divided mineral admixtures for the purpose of producing concrete that can be pumped through small diameter lines. Such concrete must not bleed readily and should remain homogeneous or plastic during its passage through the pump and the conveying pipe or hose line. The use of a sufficient quantity of suitable finely divided mineral admixture can increase the proportional volume of the matrix even though the water content is reduced, thereby increasing the capacity of the concrete for plastic deformation and reducing the rate and amount of bleeding. A discussion of these relationships is given on pp. 1507-1508 of the 1963 report[3] and also by Powers.[50] The optimum proportion of a finely divided mineral admixture to be used in a given concrete will depend on several factors, including the grading and particle shape of the fine aggregate, characteristics of the mineral admixture, and the effects that are to be produced.[49] Care should be taken in selection of a finely divided mineral admixture to improve pumpability or workability so that improvement in the properties of the fresh concrete is not secured at the cost of reduced quality of the hardened concrete.[51]

6.2.2 *Effect on strength* — The effect of a mineral admixture on the strength of concrete varies markedly with the properties of the particular admixture and with the characteristics of the concrete mixture in which it is used. For example, relatively chemically inert materials generally increase the strength of lean mixtures and decrease the strength of rich mixtures. On the other hand, cementitious materials and pozzolans contribute to strength not only because of their physical character but also by reason of their chemical composition.

For satisfactory results and for proper concrete proportioning it is important to know the use to which the concrete will be put, the probable conditions at the time of placement, and the nature of the admixture proposed for use. For example, when fly ash is used as part of the total cementitious materials, curing time to attain equal strength with concrete containing an equal amount of portland cement alone should be longer.[52] To obtain approximately equal compressive strengths at early ages between 3 and 28 days, concrete mixtures made with fly ash must have a total weight of portland cement and fly ash greater than the weight of the cement used in the comparable concrete not containing fly ash. The latter mixture will, however, contain from ¾ to 1 bag more cement per cubic yard of concrete[53,54] (1 bag per cu yd = 94 lb per cu yd = 55.7 kg per m^3).

6.2.3 *Effect on sulfate resistance* — Use of pozzolanic admixtures with portland cement in concrete generally increases resistance to aggressive attack by seawater, sulfate bearing soil solutions, and natural acid waters.[55-57] The relative improvement is greater for concrete of low cement content. The use of a pozzolan with sulfate-resisting portland cements may not increase sulfate resistance[58,59] and, if chemically active aluminum compounds are present in the pozzolan, a reduction in sulfate resistance of the concrete may result.[60] Dikeou[61] reports greatly increased sulfate resistance of concrete containing fly ash regardless of the type of cement used. The effectiveness of fly ash in improving the sulfate resistance of concrete increases as the severity of exposure to sulfates is increased.

6.2.4 *Effect on temperature rise* — At a given cement content, the use of chemically inert materials has little or no effect on the temperature rise of concrete in place. Although cementitious admixtures and, in the presence of lime and alkalies, pozzolans produce a heat of hydration that is less than that of portland cement, the temperature rise of concrete can be increased if these admixtures are used in sufficient quantity.[62] However, it is well established that in mass concrete of very low cement content, the temperature rise of concrete containing cementitious or pozzolanic admixtures is generally less than that of a comparable concrete containing portland cement as the only cementing material.

6.2.5 *Effect on expansion caused by alkali-silica reaction* — It has been reported[63,64] that almost any pozzolan when used in sufficient quantity is capable of preventing excessive expansion resulting from the alkali-silica reaction. However, the use of too small a proportion of pozzolan may actually increase detrimental effects of the alkali-silica reaction.[65] The following percentages of replacement of cement by finely divided mineral admixtures (by solid volume) have been reported[63] to afford protection against excessive expansion caused by this reaction:

a. Calcined shales: 19-29 percent

b. Volcanic glasses: 32-36 percent

c. Ground blast-furnace slag and fly ash: 39-45 percent

The reaction between cement and sand-gravel aggregate, which is widespread in the Great Plains area of the continental United States, appears to be associated with the alkali-silica reaction. The evaluation of long-time performance of test pavements indicates that pozzolans can be beneficial in reducing or eliminating map cracking and expansion resulting from this reaction.[66]

The use of pozzolans to reduce expansion caused by alkali-carbonate rock reaction in concrete has been under investigation,[72] and results to date are inconclusive but not encouraging.

6.2.6 *Effect on freezing and thawing resistance of concrete* — Of the finely divided mineral admixtures, fly ash and other pozzolans have received the most attention regarding their effect on freezing and thawing resistance of concrete. The effect of fly ash and other pozzolans on resistance of concrete to freezing and thawing and to the action of deicing chemicals during freezing depends on the proportioning of the concrete, the strength of the concrete, moisture condition of the concrete, and adequacy of air entrainment at the time of exposure.[67] Concrete containing fly ash displays the same durability characteristics as does concrete not containing fly ash provided that: (a) both mixtures are equally air-entrained and (b) both mixtures are equal in compressive strength.[68-71]

Generally, the use of a finely divided mineral admixture requires a higher proportion of air-entraining admixture to produce a given air content than that required by a comparable concrete not containing a finely divided mineral admixture. The proportion of air-entraining admixture required may vary considerably among different sources and types of finely divided mineral admixtures.

6.2.7 *Effect on permeability* — Most work on permeability of concrete containing finely divided mineral admixtures has been accomplished with pozzolans. Certain pozzolans are more effective than others in reducing permeability of concrete at early ages. However, under most conditions of service, the permeability of concrete containing any pozzolan is markedly reduced at later ages. Davis[73] concluded that in mass concrete the moderate to high replacement of portland cement by a suitable pozzolan results in a degree of watertightness not otherwise obtainable. Other investigators[52] have reported that proper use of fly ash as an admixture can reduce permeability to one-sixth to one-seventh that of equivalent concrete containing no fly ash. Part of the action of pozzolans in reducing permeability of concrete can be attributed to decreased segregation and bleeding and to any reduction of water requirement.

6.3 — Application in construction

Finely divided mineral admixtures may be used in virtually every type of concrete. These materials are generally used for one or more of the following purposes:

a. To correct a deficiency in the concrete, for example, to provide "fines" missing from the fine aggregate so as to avoid problems of workability and finishing;

b. To improve one or more qualities of the concrete, such as to increase the sulfate resistance, to reduce expansion due to alkali-silica reaction, to reduce permeability, or to decrease heat generation; and

c. Economy.

These admixtures are used in mass concrete, structural concrete, pavements, and other slabs on grade. The major emphasis has been on the following types of construction: dams, navigation locks, canal linings, tunnels, sewage works, waterworks, high-rise residential and commercial structures, and residential concrete including sidewalks, driveways, and parking areas. On the other hand, little use of finely divided mineral admixtures is found in highway concrete, although many experimental pavements contain finely divided mineral admixtures. For example, pozzolans are not widely used in portland cement concrete for pavement construction in the United States of America except in Alabama, where fly ash is specified regularly as a pozzolanic admixture. The state of Georgia now permits use of fly ash as an admixture for pavement concrete.[74,75]

Finely divided mineral admixtures are used extensively in several specialized construction and maintenance applications that involve the pumping of concrete, mortars, and grouts. These applications include grouting of ducts for post-tensioning tendons, oil well cementing, and preplaced-aggregate concrete construction.

6.4 — Evaluation and selection

The decision to use a finely divided mineral admixture from a specific source should be based upon an objective determination of the effect of alternative products upon the properties and economy of concrete for proposed types of construction. The earlier report of ACI Committee 212[3] provides helpful information in this regard. Some finely divided mineral admixtures are available at reasonable cost only in certain geographical areas. The user should determine which of the finely divided mineral admixtures that may satisfy his needs is economically available at the job location.

The cost of the concrete made with each finely divided mineral admixture under consideration should be established. Certain of these admixtures may add appreciably to the cost of the concrete by reason of increased water demand and consequent need for additional portland cement to satisfy specification requirements on water-cement ratio, strength, or other properties. On the other hand, some finely divided mineral admixtures are expected to reduce water requirement or to provide cementitious qualities themselves. In the case of pozzolans, the admixture may interact with hydration products of the portland cement to produce compounds having cementitious value. Some finely divided mineral admixtures may reduce water requirement and also produce cementitious action.

The effect of the finely divided mineral admixture on other essential qualities of concrete should be evaluated. Some of these admixtures may be helpful in reducing the heat of hydration, providing resistance to aggressive chemicals, or otherwise. Conversely, any detrimental side effects should be considered. Uniformity of the admixture from lot to lot and within shipments may be a critical factor influencing choice among alternative products inasmuch as appreciable variation in this respect will introduce problems in control of uniformity of the concrete.

6.5 — Control of purchasing and supply

When a finely divided mineral admixture in required or permitted, the specifications for the work should include purchase specifications for such admixture. Recognized standard specifications include the following:

a. Natural cement — ASTM C 10[76] and Federal Specification SS-C-185a[77]

b. Hydraulic lime — ASTM C 141[78]

c. Slag cement — ASTM C 358[23] and Federal Specification SS-C-218a[79]

d. Fly ash and raw or calcined natural pozzolan — ASTM C 618[10] and Federal Specification SS-P-570b[12]

Access to the source of the finely divided mineral admixture should be provided to the purchaser for sampling purposes.

6.6 — Storage, handling, and batching

Finely divided mineral admixtures must be stored in weathertight buildings, bins, or silos to provide protection from dampness and contamination and to minimize lumping and warehouse set.[80] Since the appearance (color and fineness) of finely divided mineral admixtures is quite often very similar to that of portland cement, storage and handling equipment should be clearly marked. If compartmented bins are used for storage of bulk hydraulic cement and finely divided mineral admixture, precaution should be taken to assure that no leakage is possible from one compartment to the other. Furthermore, compartmented bins should be inspected frequently to make certain that no leaks develop.

Finely divided mineral admixtures can be handled by the same sort or types of equipment used for conveying and transporting portland cement. These include air conveyors, screw conveyors, bucket elevators, belts, and pneumatic pumps. Some finely divided mineral admixtures, such as fly ash, have a rounded particle shape and consequently are extremely fluid when aerated. Because of this "flowability" of certain admixtures the feed of material from bin to weigh batcher may not be satisfactorily controlled by stopping and starting a transporting screw conveyor. In such cases it may be advisable to install a valve or feeder at the bin outlet to prevent the flow of material around the flight channel of the screw conveyor. The good operating practices normally associated with cement handling, such as the cleaning out of screws, pipelines, and elevator pits after each day's run, should be observed also with equipment handling finely divided mineral admixtures.

Batching of finely divided mineral admixtures must be by weight and in accordance with the requirements of ACI 614.[81] When finely divided mineral admixtures are used in bulk, a weighing sequence of cement first and admixture second is recommended. This procedure should be required when cement and finely divided mineral admixtures are weighed cumulatively on the same scale beam. Automatically controlled batching with appropriate interlocks to prevent duplication or omission is also desirable.

Finely divided mineral admixtures should be introduced into the mixer with the cement and other components of the concrete mixture. In this way, uniform quality and composition throughout the batch are assured. Finely divided mineral admixtures should not be charged into a wet mixer ahead of the other materials because of the tendency of the admixture to stick to the sides of the mixer and to the blades or fins. Likewise, finely divided mineral admixtures should not be introduced into the mixer along with the mixing water because of their tendency to ball and lump under such conditions. If the finely divided mineral admixture is introduced into the mixer after the other concrete materials, and after those materials have received some degree of mixing, it is doubtful that the admixture will ever be uniformly distributed throughout the mass. This is especially so in transit-mixed concrete.

6.7 — Proportioning of the concrete

Proportioning techniques involving the use of a finely divided mineral admixture are basically not different from those used in proportioning concrete that does not include such an admixture. Methods for selecting proportions for concrete mixtures are given in the work of ACI Committee 211.[21] Specific procedures for proportioning mixtures containing fly ash have been developed by Lovewell and Washa[53] and by Cannon.[82] Some finely divided mineral admixtures are used in such small quantities that their volume can be safely ignored. However, when finely divided mineral admixtures are used in significant proportions, as is customary with pozzolans and cementitious materials, their solid volume must be taken into account in proportioning calculations. Since finely divided mineral admixtures are usually as fine or finer than portland cement, they should usually be regarded as a part of the cement paste matrix in determining the percentages of fine and coarse aggregate.

The effect of the proposed admixture on the mixing water requirement should also be known. Some finely divided mineral admixtures cause a major increase in water requirement, other admixtures of this type have little or no effect on water requirement, whereas certain finely divided mineral admixtures typically reduce the water requirement of concrete in which they are used. In general, relatively chemically inert finely divided mineral admixtures have no direct effect on the required amount of portland cement in the concrete mixture other than the fact that they may increase or decrease the total water requirement of the concrete, thereby making necessary an adjustment in the cement content. Cementitious admixtures and pozzolans on the other hand not only affect the water requirement of the concrete and thereby the cement content but, because of their properties, they are often considered as a part of the cementing material.[83] They are usually used in the range 15 to 35 percent by weight of the total cementitious medium of the concrete, depending on the purpose for which the concrete is to be employed and the specifications for the work.[53]

Relatively chemically inert finely divided mineral admixtures are sometimes used in concrete to make up for a deficiency of fines in the fine aggregate. In such an application, the admixture is considered to be a part of the fine aggregate fraction and its presence should have no effect on the cement content unless the mixing water requirement is changed.

As is true also for concrete mixtures that do not contain an admixture, trial mixtures should be prepared and tested to validate the calculated proportioning of concrete containing finely divided admixtures.

6.8 — Control of the concrete

Concrete containing a mineral admixture should be measured, mixed, and placed in accordance with ACI 614.[81] In charging of the mixer, care should be taken to avoid loss of the admixture, since some materials of this type are of very high surface area and may be of low specific gravity so as to be especially likely to be blown away. Mixing time to achieve uniformity of composition of the concrete may differ from that of equivalent concrete not containing the mineral admixture; the minimum mixing time required should be determined by testing of the concrete for uniformity using procedures such as those described in Section 9 of ASTM C 94-69[84] and Section 10 of ACI 614.[81]

Variation in physical or chemical properties of the admixture, even though within the tolerance of usual specifications, may cause appreciable variation in properties of the concrete, such as change of water content for given consistency and of the requirement for air-entraining admixture for given air content. These variations commonly are reflected in change of time of setting, early strength development, and color of the concrete in place.

Such circumstances can be minimized by appropriate testing of shipments of the admixture before use of the successive lots in concrete. For example, to accommodate a highly variable fly ash, at a particular project, sampling of each carload and testing for loss on ignition, fineness, and air entrainment in mortar was required.[85] These data permitted estimation of the proportion of air-entraining admixture that would be required to achieve the specified air content of the concrete supplied to the work.

Control of color is of increasing significance for concrete that will be exposed to public view; contribution of a finely divided mineral admixture to such variation can be minimized by using in given construction an admixture from one source only and successive lots may be compared against an approved standard. Procedures for such control are discussed by Hyland.[86]

The quality and uniformity of concrete containing a finely divided mineral admixture can be monitored by use of strength tests of concrete delivered to the work. Procedures described in ACI 214[87] apply. Standard tests of control cylinders, made in accordance with ASTM C 31[88] and ASTM C 39, provide an evaluation of the quality and uniformity of the concrete as a material, but not a measure of the properties and uniformity of the concrete in place in the construction. Information on setting time and strength gain of concrete in place or in precast units can be secured by testing of specimens cured under conditions simulating those affecting the work, such as low ambient temperatures or steam curing.

Adequate control of concrete, with or without a mineral admixture, requires a consistent program of inspection, such as that described in ACI 311.[89] Properly executed inspection with cooperation between the owner and his representatives and the contractor will give assurance that the concrete will be more uniform and will more consistently meet the requirements of the work, and any problems that may arise will be less likely to proceed to a critical stage before they are rectified.

Concrete containing a finely divided admixture in such proportion as to substantially reduce cement content and increase water-cement ratio, may show retarded setting and unusually slow development of strength at low ambient temperatures unless the concrete-making materials are heated and the concrete in place is protected from undue cooling, in accordance with stipulations of the ACI 306 standard.[31] Such problems are less likely to be significant in mass concrete where retardation usually is tolerable and may be desirable, but in structural concrete and in pavements, floors, and slabs on grade requiring finishing in accordance with a rapid rate of construction, the circumstances may dictate a reproportioning of the concrete mixture, use of high-early-strength cement as the portland cement component, or addition of an accelerating chemical admixture, preferably with heating of the concrete-making materials.

CHAPTER 7 — MISCELLANEOUS ADMIXTURES

7.1 — Gas-forming admixtures

7.1.1 *Introduction* — The void content of concrete can be increased by the use of admixtures that generate gas bubbles in the fresh mixture during and immediately following placing, prior to development of set in the cement paste matrix. Such materials are added to the concrete mixture to counteract settlement and bleeding thus causing the concrete to more nearly retain the volume in which it was cast.

7.1.2 *Materials* — Admixtures which produce these effects are hydrogen peroxide which releases oxygen; metallic aluminum[90,91] which releases hydrogen; and certain forms of activated carbon from which adsorbed air is liberated. Only aluminum powder is used extensively for this purpose as a concrete admixture. The unpolished powder is preferred although the polished powder may be used when a slower reaction is desired. The rate and duration of the gas release depends on many conditions, including composition of the cement and temperature, water-cement ratio, fineness and particle shape of aluminum powder, and the effectiveness of the treatment is controlled by the duration of mixing, handling and placing operations relative to the speed of gas generation. The addition rate may vary from 0.005 to 0.02 percent by weight of cement under normal conditions although larger quantities may be used to produce low strength cellular concrete. Approximately twice as much aluminum is required at 40 F (4 C) as at 70 F (21 C) to produce the same amount of expansion. Because of the very small quantities of aluminum powder generally used (about 1 teaspoonful per bag of cement), and because it has a tendency to float on the mixing water, it is generally premixed with fine sand, cement, or pozzolan, or incorporated in commercially available admixtures having water-reducing, set-retarding effects.

7.1.3 *Effect* — The release of gas, when properly controlled, causes a slight expansion of freshly mixed concrete. When such expansion is restrained there will be an increase in bond to horizontal reinforcing steel without excessive reduction in strength. Too much gas producing material may produce large voids seriously weakening the matrix. The effect on strength depends to a considerable extent on the degree to which the tendency of the mixture to expand is restrained. It is therefore important that confining forms be tight and adequately closed. Gas forming agents will not overcome shrinkage after hardening caused by drying or carbonation. In cold weather it may be necessary to speed up the rate of gas generation by the addition of alkaline materials such as sodium hydroxide, hydrated lime, or trisodium phosphate. This may be done to ensure sufficient gas generation before the mixture has set.

7.2 — Grouting admixtures

7.2.1 *Introduction* — Many admixtures used for specific purposes in concrete as well as materials to impart special properties in a grout have been suggested as grouting admixtures. Such grouts are used primarily in cementing oil wells where high temperatures and pressures may be encountered and pumping distances are considerable.

7.2.2 *Materials* — Retarders, as described in Chapter 5, are useful in delaying set. Materials such as gels, clays, pregelatinized starch, and methyl cellulose have been suggested to prevent the rapid loss of water from the grout.[92] Bentonite clays may be used to reduce slurry weights and materials such as barite and iron filings used to increase the weight.[92] Thickeners, such as natural gums, may be added to prevent settlement of heavy constituents of the grout. Special applications may find other admixtures such as accelerators and gas-forming materials as described in other sections to be suitable. Since some special cements, particularly those used for cementing oil wells, may contain an agent or agents of the type described, tests should be conducted to determine compatibility of any admixture with the cement to be used.

7.2.3 *Effect* — Retarders may be used to keep a grout fluid at temperatures up to 400 F (200 C) and pressures as high as 18,000 psi (127 kgf/cm^2) for one or more hours. Since this is a highly specialized field requiring properties not encountered with ordinary concreting operations, tests must be made to determine the addition rates of the admixtures required to develop the desired properties.

7.3 — Expansion-producing admixtures

7.3.1 *Introduction* — Admixtures, which during the hydration period of the concrete expand themselves or react with other constituents of the concrete to cause expansion, are used to minimize the effects of drying shrinkage. They are used in both restrained and unrestrained concrete placement.

7.3.2 *Materials* — The most common admixture for this purpose is finely divided or granulated iron, and chemicals to promote oxidation of the iron. The use of admixtures for this purpose is generally limited to relatively small projects where varying degrees of expansion are desired. Expansive cements are used on large projects where a predetermined uniform degree of expansion is required.[93] For additional information regarding these cements refer to the report of ACI Committee 223.[2]

7.3.3 *Effect* — The controlled expansion produced by those materials may be of about the same magnitude as the drying shrinkage expected at later ages or it may be greater. For a given application, the extent of expansion and the time interval during which it takes place are very important and must be under control for the most satisfactory results. For unrestrained concrete, the expansion must not take place before the concrete gains sufficient strength to be stressed in tension rather than disrupted by the expanding forces. For restrained applications, the concrete must be strong enough to withstand the compressive stresses developed. It is reported that restraint in only one direction is required[94] to achieve some degree of compression in the other two orthogonal directions.

7.4 — Bonding admixtures

7.4.1 *Introduction* — Admixtures specifically formulated for use in portland cement mixtures to enhance bonding properties generally consist of an organic polymer emulsion.[95-101] Usually they increase the air content of the mixture in which they are used.

7.4.2 *Materials* — Since the products of portland cement hydration are alkaline in nature and contain calcium ions, the bonding emulsion must be so formulated as to be stable under these environmental conditions. Some emulsion systems are unstable in acid environments, some in alkaline environments, and some in the presence of calcium ions. An unstable emulsion will coagulate in the mixture rendering it unsuitable for use. In general, emulsions of synthetic materials are more universally stable than those made of natural rubbers.

7.4.3 *Function* — When used as admixtures in quantities normally recommended by the manufacturers, 5 to 20 percent by weight of the cement, bonding materials cause the fresh concrete to be sticky. This is partially due to air-entrainment and partially to the inherent nature of the admixture.

Water is necessary to hydrate the portland cement of the cement-polymer system but the polymer component becomes effective only when the emulsion is broken through a drying out process. The polymer emulsion carries a considerable quantity of water into the mixture, the water being released to the cement during the hydration process. At the same time, this release of water sets the emulsion. Hence, moist curing is not only unnecessary but undesirable since the emulsion will not have an opportunity to dry and develop the desired strength.

The compressive strength of grouts, mortars, and concrete made with these materials may be greater or less than that of non-admixtured mixtures of the same cement content which are moist cured, depending on the material used.[102] However, the increase in bond and flexural strength far outweighs the possible disadvantage of compressive-strength reduction.

7.4.4 *Limitations* — Some types of polymers will soften in the presence of water, and these types should not be used in areas that will be in contact with water. The ultimate result obtained with a bonding admixture is only as good as the surface to which the mixture is applied. The surface must be clean, sound, and free from foreign matter such as paint, grease, and dust. Bonding materials are particularly adaptable for use in patching operations where feathered edges are desired. A thin application of grout or mortar containing the bonding admixture develops a higher bond strength than a thick application. When properly applied and cured, such a bond is often stronger than the materials that are being joined.

7.5 — Coloring admixtures

7.5.1 *Introduction* — Pigments specifically prepared for use in concrete and mortar are available both as natural and synthetic materials. They are formulated to produce adequate color without materially affecting the desirable physical properties of the mixture.

7.5.2 *Materials* — The pigments listed below may be used to obtain a variety of colors.

Shades of color	Pigment
Grays to black	Black iron oxide Mineral black Carbon black
Blue	Ultramarine blue Phthalocyanine blue
Bright red to deep red	Red iron oxide
Brown	Brown iron oxide Raw and burnt umber
Ivory, cream, or buff	Yellow iron oxide
Green	Chromium oxide Phthalocyanine green
White	Titanium dioxide

7.5.3 *Effect* — The addition rate of any pigment normally should not exceed 10 percent by weight of the cement.[103,104] Natural pigments, usually not ground as fine and often not as pure as synthetic materials, generally do not produce as intense a color per unit of addition.

Addition rates below 6 percent generally have little or no effect on the physical properties of the fresh or hardened concrete. Larger quantities may increase the water requirement of the mixture to such an extent that the strength and other properties such as abrasion resistance may be adversely affected.

The addition of a nonmodified carbon black will increase considerably the rate of use of an air-entraining admixture necessary to provide air content sufficient for proper resistance of the concrete to freezing and thawing.[105] Most carbon blacks available for coloring concrete do, however, contain air-entraining materials in sufficient quantity to offset the inhibiting effect of the carbon black.

7.5.4 *General requirements* — Suitable coloring admixtures should meet the following requirements:

1. Color fastness to sunlight
2. Chemical stability in the presence of alkalinity produced by the cement reaction
3. Color stability when exposed to autoclaving
4. No adverse effects on the setting time or strength development of the concrete

Information regarding the properties of pigments related to the first three items may be found by referring to Payne.[106] Some pigments offered for coloring concrete are less effective than others. It is difficult to get strong blue or green colors. Efflorescence often causes conditions which are mistaken for fading. ASTM is currently developing specifications for pigments for coloring concrete.

7.6 — Flocculating admixtures

The addition of certain chemical admixtures to cement paste, mortar, or concrete, has the ability to alter some of the properties of the freshly mixed material. Synthetic polyelectrolytes have been used as flocculating admixtures. The published reports[107,108] indicate that these materials increase the bleeding rate, decrease the bleeding capacity, reduce flow, increase cohesiveness, and increase green strength.

7.7 — Fungicidal, germicidal, and insecticidal admixtures

7.7.1 *Introduction* — Certain materials have been suggested as admixtures for concrete or mortar to impart fungicidal, germicidal, and insecticidal properties. The primary purpose of these materials is to inhibit and control the growth of bacteria and fungus on concrete floors and walls. They may not always be completely effective.

7.7.2 *Types of materials* — The materials that have been found to be most effective are:

Polyhalogenated phenols[109,110]
Dieldrin emulsion[111]
Copper compounds[112,113]

7.7.3 *Effect* — Addition rates vary from 0.1 to 10 percent by weight of the cement depending on the concentration and composition of the chemical. The higher addition rates, above 3 percent, may have an adverse effect on the strength of the concrete. The effectiveness of these materials, particularly the copper compounds, is reported to be of a temporary nature. This will probably vary with the type of wear and cleaning methods employed.

7.8 — Dampproofing admixtures

7.8.1 *Introduction* — Some concrete dams, retaining walls, tanks, and other structures show evidence of leakage. Usually, such leakage is the result of faulty production and placement of concrete, or it is due to cracks in the structure. When properly proportioned concrete mixtures are used and placed with high-class workmanship under qualified inspection, the concrete in a structure should be virtually impermeable, although leakage may still occur through cracks.

The term "dampproofing" implies prevention of water penetration of dry concrete, or stoppage of transmission of water through unsaturated concrete. However, admixtures have not been found to produce such effects; the term has come to mean a reduction in rate of penetration of water into dry concrete, or in rate of transmission of water through unsaturated concrete from the damp side to the dryer side.

An admixture described as a dampproofer may have some such beneficial effect on the properties of fresh concrete not directly indicated by the name. For example, it may promote entrainment of air and thus may properly be considered an air-entraining admixture. This section deals with those aspects directly implied by the term dampproofing. This implies an effect on the properties of hardened concrete, apart from whatever effect the admixture might have on freshly mixed concrete. This discussion therefore deals with the possible effects of such agents on the properties of hardened concrete.

7.8.2 *Materials* — Admixtures for dampproofing include soaps, butyl stearate, and certain petroleum products.[114-119]

1. The soaps comprise salts of fatty acids, usually calcium or ammonium stearate or oleate. The soap content is usually 20 percent or less the remaining being calcium chloride or lime. Total soap added should not exceed 0.2 percent by weight or cement. Soaps cause entrainment of air during mixing.

2. Butyl stearate has an action like soap in that it provides a water-repellent effect but does not entrain air. It is added as an emulsion with the stearate being 1 percent by weight of the cement. Reports indicate better results than from use of soap as a water repellent and the effect on the strength is negligible.

3. Among petroleum products are mineral oils, asphalt emulsions, and certain cutback asphalts. Heavy mineral oil is effective in rendering concrete water repellent and in reducing its permeability. It must be a fluid petroleum product having a viscosity of SAE 60, with no fatty or vegetable oils. Oil added at rate of 5 percent

by weight of cement is only slightly detrimental to concrete strength and has proven to be effective under pressure.

4. There is a group of miscellaneous materials sometimes available on the market. All of these are usually detrimental to concrete strength and none are truly dampproofers. These include:

a. Barium sulfate and calcium and magnesium silicates

b. Finely divided silica and naphthalene

c. Colloidal silica and a fluosilicate

d. Petroleum jelly and lime

e. Cellulose materials and wax

f. Silica and aluminum

g. Coal tar cut with benzene

h. Sodium silicate

7.8.3 *Effects* — Dampproofing admixtures, by reducing penetration of the visible pores, may retard penetration of rain into concrete block made of nonplastic mixtures. Test data show that they reduce also the rate of penetration of moisture into the micropores of dry concrete, but there is no indication that there are comparable effects on the transmission of moisture through unsaturated concrete, except when the concrete contains paste having relatively high porosity. A paste of high porosity results from low cement content and correspondingly high water-cement ratio, lack of curing, or from both factors. If the concrete has a sufficiently low porosity such as that obtained by producing a well-cured paste having a water-cement ratio not over 0.6 by weight, dampproofing agents give no appreciable improvement.

The Building Research Advisory Board[120] reported that in the opinion of the majority of 61 interrogated observers, dampproofing admixtures are not " . . . effective or acceptable in controlling moisture migration through slabs-on-ground." It also reported that a special advisory committee to the Building Research Advisory Board reached the following conclusion on the basis of data from tests on moisture transmission through unsaturated concrete slabs: "The Committee does not find adequate data to demonstrate the effectiveness of any admixture to reduce the transmission of moisture through concrete slabs-on-ground in a manner sufficient to replace either a vapor barrier or granular base, or both, under conditions where such protection would be needed."

7.9 — Permeability-reducing admixtures

7.9.1 *Discussion* — The term permeability usually refers to a coefficient giving the rate at which water is transmitted through a saturated specimen of concrete, under an externally maintained hydraulic gradient. Admixtures of the kinds discussed in Section 7.8 do not reduce the coefficient of permeability of the saturated concrete. However, mineral powders, properly proportioned, reduce the permeability of mixtures in which the cement content of the paste is relatively low. Under conditions where this effect is obtained, there is usually also a reduction in the amount of water per cubic yard, and thus a small reduction in porosity.

The reduction of total water content by means of a water-reducing admixture should reduce the total porosity slightly, but there are no adequate data to demonstrate that permeability is thereby reduced materially.

Accelerating admixtures such as calcium chloride increase the average rate of hydration and thereby reduce the length of time required for a concrete mixture to attain a given fraction of its ultimate degree of impermeability. However, any advantage attained this way is likely to be temporary since, if conditions are such that water is being transmitted through the concrete, they are also conducive to continued hydration of cement.

7.10 — Chemical admixtures to reduce alkali-aggregate expansion

7.10.1 *Introduction* — As early as 1950, reports began to appear on admixtures to reduce expansion caused by alkali-aggregate reaction. Since that time there has been relatively little new information added in the form of meaningful research or field practice.[3]

7.10.2 *Materials* — Two salts, lithium and barium, and certain air-entraining and some water-reducing, set-retarding admixtures have all been reported to produce reductions in expansion of laboratory mortar specimens. Outstanding reductions have been obtained in such specimens using 1 percent additions of the lithium salts and 2 to 7 percent additions by weight of cement for certain barium salts. Salts of proteinaceous materials and water-reducing, set-retarding materials have shown moderate reductions in expansion. Data on the protein air-entraining admixtures are based on the use of 0.2 percent by weight of cement.

The lithium salts are very expensive. Where barium salts are used, they must be incorporated in the powdered clinker rather than in the cement due to their lack of solubility. Some laboratory reports indicate the use of calcium chloride with the barium salts to counteract strength loss.

Air entrainment, regardless of the admixture used, has been shown to lower slightly the expansion.

The use of pozzolans to reduce expansion caused by alkali-aggregate reaction has been widely studied and reported (see Chapter 6).

7.10.3 *Effects* — Only very limited laboratory mortar bar test data are available; therefore no recommended practices are herein presented. The above summarizes the information now available. Obviously any user of these materials should make additional tests before proceeding to practical field use.

7.11 Corrosion — Inhibiting admixtures

7.11.1 *Introduction* — Many investigators have studied the corrosion of iron and steel with particular reference to protective coatings. It has been found that concrete furnishes ample protection to the steel embedded in it, except in certain cases in which infiltrating or percolating waters find a way through the concrete, removing or carbonating the calcium hydroxide.[121,122]

The problem of corrosion of reinforcing steel in concrete has generally been limited to concrete exposed to saline or brackish waters (or those containing deicing chemicals) or soils containing chlorides from which chlorides can reach the steel either by diffusion through the concrete or by entrance through cracks. Probably because it was recognized that concrete itself was a good protective coating, and because the work with paints indicated that inhibitors such as chromates would not provide protection under conditions where chlorides could enter the concrete, there is little information in the technical literature pertaining to the use of corrosion-inhibiting admixtures in concrete.

7.11.2 *Materials* — The use of sodium benzoate at a rate of 2 percent in the mixing water or a 10 percent benzoate-cement slurry painted on reinforcement, or both, are described as effective.[123,124] Analysis showed that the sodium benzoate remained in the concrete after 5 years exposure. It is also an accelerator of compressive strength.

Calcium lignosulfonate has been found to reduce the rate of corrosion of steel in concrete containing calcium chloride.[125]

Sodium nitrite has been investigated by Moskvin and Alekseyev[126] as an inhibitor of corrosion of steel in autoclaved products. These authors suggest that the high alkalinity, which is normally present in concrete and which serves to passivate the steel, may be considerably reduced by autoclave treatment especially when siliceous admixtures are present. Two to three percent sodium nitrite by weight of cement was found to be an efficient inhibitor under these conditions. Sarapin[127] found by storage tests that 2 percent sodium nitrite was effective in preventing corrosion of steel in concrete containing calcium chloride.

Low solubility salts such as certain phosphates or fluosilicates and fluoaluminates are beneficial according to limited reports. Dosage should be limited to 1 percent by weight of cement.

In the manufacture of certain concrete products containing steel, it might be desirable to accelerate the rate of strength development by use of both a chemical accelerator and heat, the latter usually in the form of steam at atmospheric pressure. When calcium chloride is used as the accelerator in this type of curing, the rate of corrosion of the steel has been found in laboratory studies to be accelerated. However, Arber and Vivian[128] found that certain compounds which contain an oxidizable ion, such as stannous chloride, ferrous chloride, and sodium thiosulfate, act as accelerators as does calcium chloride, but also appear to cause less corrosion than the latter. Stannous chloride appeared to be the best of the products tried and 2 percent of the salt by weight of cement was more effective than 1 percent, and as effective as greater amounts, both from the standpoint of acceleration and resistance to corrosion. Stannous chloride oxidizes to stannic chloride when in solution and may also oxidize in situ where lean relatively permeable concrete furnishes inadequate protection against access to oxygen. For effective use, the salt must be added to the concrete in the stannous form and a dense concrete must be used.

7.11.3 *Effects* — Warnings have been sounded against the use of inhibitors. For example, the South African National Building Research Institute[129] made the following statement, "Integral Additives; Although certain inert and reactive materials have shown promise, their use cannot be recommended at this stage, as insufficient evidence of their effectiveness or possible disadvantages is available." Evans[130] stated: "A beneficial effect from the inhibitor addition might reasonably be expected if the steel surface is clean and chlorides absent, but under such conditions there is unlikely to be serious trouble even without added inhibitor. Where rust particles occur there is a risk that the rust, preventing the inhibitor from reaching the metal below it, may establish the combination of small anodes and large cathodes . . . making matters worse."

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This report was submitted to letter ballot of the committee which consists of 21 members; of whom 20 returned ballots and all voted affirmatively; one member of the committee is deceased.
For discussion see ACI Journal, *Proceedings* V. 69, No. 3, Mar. 1972, pp. 189–190.

ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction, and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be incorporated directly into the Project Documents.

ACI 212.1R-63

From ACI JOURNAL, Nov. 1963

Admixtures for Concrete

Reported by ACI Committee 212

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This third report of ACI Committee 212, Admixtures for Concrete, updates the previous reports of 1944 and 1954. In this report admixtures are classified in 15 groups according to type of materials constituting the admixtures, or to the characteristic effects of their use. Where an admixture possesses properties identifiable with more than one group it is discussed with the group that describes its most important effect on concrete. Types of admixtures discussed are: (1) accelerating; (2) water-reducing and set-controlling; (3) grouting; (4) air-entraining; (5) air-detraining; (6) gas-forming; (7) expansion-producing; (8) finely divided mineral; (9) damp-proofing and permeability-reducing; (10) bonding; (11) alkali-aggregate-expansion-reducing; (12) corrosion-inhibiting; (13) fungicidal, germicidal, and insecticidal; (14) flocculating; and (15) coloring. An extensive list of references is included.

Key words: accelerating admixture; ACI committee report; admixture; air-detraining admixture; air-entraining admixture; alkali-aggregate reaction; bonding admixture; chemical admixture; coloring admixture; concrete; corrosion-inhibiting admixture; damp-proofing admixture; expansion-producing admixture; finely divided mineral admixture; flocculating admixture; fungicidal admixture; gas-forming admixture; germicidal admixture; grouting admixture; heat evolution; insecticidal admixture; permeability-reducing admixture; set-controlling admixture; shrinkage; sulfate resistance; water-reducing admixture.

PREFACE

This is the third report of Committee 212, on Admixtures for Concrete, the first and second having appeared in 1944[1] and 1954,[2] respectively. In general, the method of presentation is similar to that of the previous reports and some of the sections are essentially unchanged from the last edition. The recent more widespread use of admixtures and the availability of new information on their properties and applications are the reasons for this revision.

The 1959 American Society for Testing and Materials Symposium on the effect of water-reducing admixtures and set-retarding admixtures on properties of concrete[3] is an indication of the growing interest in this particular group of admixtures, as is the estimate that they are now employed in over 25,000,000 cu yd of concrete in the United States each year. Accordingly, the brief mention of this group in the 1954 report has been amplified in the present one.

The section on air-entraining admixtures has been revised to include information on the air void characteristics of air-entrained concrete and the use of air entrainment in structural lightweight aggregate concrete. The sections on cementitious materials, pozzolanic materials, and on theoretical considerations on the use of pozzolanic materials and mineral powders as admixtures have been rewritten and combined. More detailed information on fly ash has been added. New sections on coloring admixtures; on fungicidal, germicidal, and insecticidal admixtures; on bonding admixtures; on air-detraining admixtures; on corrosion-inhibiting admixtures; and on flocculating admixtures have been added. The discussion of "damp-proofing" and permeability-reducing admixtures has been revised and shortened, while several sections have been omitted.

One major section omitted from the current report is that on workability agents. The effects on workability of the various admixtures are discussed at appropriate locations throughout the report. The report includes a number of recommendations relating to the use of admixtures.

CONTENTS

INTRODUCTION

An admixture is defined by the ASTM in the Standard Definitions of Terms Relating to Concrete and Concrete Aggregates (C 125)* as: "A material other than water, aggregates, and portland cement (including air-entraining portland cement and portland blast-furnace slag cement) that is used as an ingredient of concrete and is added to the batch immediately before or during its mixing."

An admixture is used to modify the properties of the concrete in such a way as to make it more suitable for the work at hand. Use of a suitable admixture can impart certain desirable characteristics which cannot be secured by other methods, or not as economically. In other instances, the desired objectives may be achieved most economically by changes in composition or proportions of the concrete mixture rather than by the use of an admixture. Admixtures are no substitute for good concreting practices. An admixture should be employed only after appropriate evaluation of its effects, if necessary by use with the particular concrete and under conditions of use intended. Admixtures should be required to conform with applicable ASTM or other relevant specifications. In using any admixture, careful attention should be given to the instructions provided by the manufacturer of the product.

Some of the more important purposes for which admixtures have been used are:

1. Improvement of workability
2. Acceleration of the rate of strength development at early ages
3. Increase in strength
4. Retardation or acceleration of initial setting
5. Retardation or reduction of heat evolution
6. Modification in rate of, and capacity for, bleeding
7. Increase in durability or in the resistance to special conditions of exposure including application of ice-removal salts
8. Control of alkali-aggregate expansion
9. Decrease in capillary flow of water
10. Decrease in the permeability to liquids
11. Production of cellular concrete

*See the most recent *ASTM Standards*, American Society for Testing and Materials, Philadelphia, Pa.

12. Improvement of penetration and pumpability, and the reduction of segregation in grout mixtures
13. Prevention of settlement, or creation of slight expansion, in concrete and mortar used for filling blockouts or other openings in concrete structures, and in grout for seating machinery, columns or girders, or for filling post-tensioning cable ducts or voids in preplaced aggregate
14. Increase in bond of concrete to steel reinforcement
15. Increase in bond between old and new concrete
16. Production of colored concrete or mortar
17. Production of fungicidal, germicidal, and insecticidal properties
18. Corrosion inhibition

In considering the effects of admixtures in concrete it is pointed out that: (1) a change in type of cement or amount of cement used, or a modification of aggregate grading or mixture proportions may be desirable; (2) many admixtures affect more than one property of concrete, sometimes affecting desirable properties adversely; (3) the effects of some admixtures are significantly modified by such factors as wetness and richness of the mixture, by aggregate grading, and by character and length of mixing; (4) the specific effects of some admixtures vary with the type, quantity, and properties of cement used; and (5) accordingly, some specific effects of an admixture usually can not be predicted accurately prior to testing.

ECONOMIC ASPECTS OF THE USE OF ADMIXTURES

Use of an admixture may increase or decrease the cost of the concrete. The effect of a given admixture can sometimes be obtained, at least in some degree, by other means or by other admixtures. Whenever possible, the cost of an admixture should be compared with that of alternative materials or methods for getting the desired result. Conversely, the economic gains possible through decreased construction costs by the use of admixtures should also be considered.

In evaluating an admixture, its effect on the volume of a given batch should be noted. If adding the admixture changes the volume, as is often the case, the change in the properties of the concrete will be due not only to direct effects of the admixture but also to the changes in the amounts (per unit volume) of the original ingredients. If the use of the admixture increases the volume of the batch, the admixture must be regarded as effecting a displacement either of part of the original mixture or of one or another of the basic ingredients — cement, aggregate, or water. All such changes in the composition of a unit volume of concrete must be taken into account when testing the direct effect of the admixture itself, and in estimating the cost of the use of an admixture.

The cost of handling an extra ingredient, and any effect the use of the admixture may have on the cost of transporting, placing, and finishing the concrete should be taken into account. Frequently an admixture permits use of less expensive construction methods or even of less expensive structural design so as to offset any increase of cost of materials attending the use of an admixture. For example, novel and economical designs of structural units have been predicated on use of retarding admixtures that permit placing of concrete over extended periods in homogeneous units of large size and great volume, thus minimizing need for training and water-reducing admixtures commonly makes possible the attaining of required physical properties of lightweight concrete at lower unit weight.

The evaluation of the cost of any given admixture should be based on the results obtained with the particular concrete in question under conditions simulating those expected on the job. This is highly desirable since the results obtained are influenced to an important degree by the characteristics of the cement and aggregate and their relative proportions, as well as by temperature, humidity, and curing.

CLASSIFICATION OF ADMIXTURES

In this report, admixtures are classified in 15 groups, according to type of materials constituting the admixtures, or to the characteristic effects of their use. Information to characterize each group is presented along with brief statements of the general purposes and expected effects of the use of materials of each group. The wide scope of the admixture field, the continual entrance of new or modified materials into this field, and the variations of effects with different concreting materials and conditions, precludes a detailed listing of commercial admixtures and their effects on concrete. The groups are listed in the Contents.

Commercial admixtures may contain materials that separately would belong in two or more of these groups. For example, a water-reducing admixture may be combined with an air-entraining admixture. Those types of admixtures possessing properties identifiable with more than one group are considered in the discussion following as belonging to the group that describes their most prominent or important effect on the concrete.

ACCELERATING ADMIXTURES

General

Accelerating admixtures are added to concrete either (a) to increase the rate of early strength development, (b) to shorten the time of setting, or (c) for both purposes. The use of accelerators often permits better scheduling of the work. The benefits of an increase in the rate of early strength development may include: (1) earlier removal of forms, (2) reduction of required period of curing and protection, (3) earlier placement in service of a structure or a repair to a structure, (4) partial or complete compensation for the effects of low temperatures on rate of strength development. The benefits of a reduced time of setting may include: (1) early finishing of surfaces, (2) reduction of pressures on forms or of length of time during which forms are subjected to hydraulic pressure, (3) more effective plugging of leaks against hydraulic pressure.

Chemicals which accelerate the hardening of mixtures of portland cement and water include some of the soluble chlorides, carbonates, silicates, fluosilicates, and hydroxides,[4] and also some organic compounds such as triethanolamine.[5,6] Calcium aluminate cements and finely-ground hydrated portland cement have also been advocated.

Some of the soluble chlorides, particularly calcium chloride,[7,8,9] and to a much lesser extent triethanolamine have general applicability as admixtures in concrete. Some of the other materials are suitable only for use in the preparation of "quick set" cements.

Available information permits only brief discussion of accelerators other than calcium chloride.

Triethanolamine is used in relatively small quantities, usually in combination with other materials. The use of stannous chloride, ferrous chloride, and sodium thiosulfate[10] is mentioned in the section on corrosion-inhibiting admixtures.

It has been reported that the time of setting of portland cement may be shortened, through varying degrees, by the use of 5 to 20 percent by weight of the portland cement, of calcium aluminate cement.[11,12] Also the setting time of calcium aluminate cements may be greatly shortened by additions of either lime or portland cement. The compressive strength at 1 day or more of neat cement, mortar, and concrete prepared with mixtures of portland and calcium aluminate cements will generally be materially lower than those obtained with either of the two cements alone. Drying shrinkage and swelling in water are higher for such mixtures and the durability may be adversely affected.[13]

The "seeding" of portland cement concrete with 2 percent by weight of the cement with finely ground, fully hydrated portland cement has been reported to be equivalent to the use of 2 percent of calcium chloride with the additional advantage of increases in 90-day strengths of 20 to 25 percent and no increase in drying shrinkage.[14,15] The effects of seeding and calcium chloride addition are said to be additive.

Some of the chemicals listed above are used to produce fast-setting mortars for sealing leaks and for other special purposes. Various proprietary compounds are available. They are furnished in liquid or powder form to be mixed with cement or cement and sand. Used undiluted with cement, setting times of as little as 15 to 30 sec are obtained. There are ready-to-use mixtures of accelerator, cement, and sand that will have an initial set in 1 to 4 min, and a final set of 3 to 10 min. Mortars thus prepared are employed to seal leaks in below-grade structures, for patch work, and for emergency repair work. The ultimate strength of such mortar will usually be much lower than if no accelerator had been used.

Accelerators purchased for general applicability in concrete should meet the requirements of ASTM C 494 "Tentative Specifications for Chemical Admixtures for Concrete, Type C," and calcium chloride should also meet the requirements of ASTM D 98, "Tentative Specifications for Calcium Chloride."

Calcium chloride

Calcium chloride is available in two forms. Regular flake calcium chloride, ASTM D 98 Type 1, contains a minimum of 77 percent $CaCl_2$. Concentrated flake, pellet, or granular calcium chloride, ASTM D 98 Type 2, contains a minimum of 94 percent of $CaCl_2$. Calcium chloride can generally be used safely[7,16] in amounts up to 2 percent by weight of the cement. Larger amounts may be detrimental and except in rare instances provide little additional advantage. The benefits of the use of calcium chloride are usually more pronounced when it is employed in concrete with a mixing and curing temperature below 70 F. At high mixing and curing temperatures long-term strength, especially flexural strength, may decrease, and shrinkage and cracking may increase.

Laboratory tests have indicated that most increases of compressive strengths of concrete resulting from the use of 2 percent of calcium chloride by weight of cement are in the range of 400 to 1000 psi at 1 through 7 days for 70 F curing. At 40 F curing the increases in strengths obtained at 1 and 7 days with calcium chloride are in the same range as that for 70 F curing. The increase in strength usually reaches its maximum in 1 to 3 days and thereafter generally decreases. At 1 year, some increase is still evident in concrete made with most cements. The specific effect of the use of calcium chloride varies, however, for different cements as is indicated by the range of strength increases cited above for the early ages.

The relative increase in flexural strength of concretes resulting from the use of 1 or 2 percent of calcium chloride is not as great as the increase in compressive strength. Calcium chloride increases the flexural strength at 1 and 3 days, but decreases the flexural strength at 28 days or at later ages.[7,16]

Flexural strengths of concretes containing 1 to 2 percent calcium chloride are usually increased by 40 to 90 percent at 1 day and by 5 to 35 percent at 3 days, respectively, when moist cured at 70 F, over the strengths of similar concrete without the admixture. At 28 days, decreases of up to 12 percent have been reported from laboratory tests of moist-cured concrete.

The use of 1 percent calcium chloride by weight of the cement is sufficient in most cases to accelerate setting and increase strength sufficiently for cold weather concreting, with the understanding that cold weather protection is provided.[17] The selection of the optimum amount should be based on the type of cement, the temperature of the concrete, and the ambient air temperature.

The foregoing discussion on strength has been limited to the use of calcium chloride with portland cement in concrete. There are insufficient data on the use of calcium chloride with portland blast-furnace slag cement or other blended cements to justify any conclusions on the effects of their combination in concrete. Calcium chloride should not be used with calcium aluminate cements.

Other effects resulting from the use of calcium chloride include a small increase in the workability of the fresh concrete, increase in air content and average size of air voids when used with air-entraining agents, and early commencement of the stiffening with some cements, and accordingly, a reduction in bleeding.

Drying shrinkage generally, but not always, has been found to be increased when calcium chloride is used. Differences observed by various investigators may be due to differences in curing procedure. The rate of heat evolution is increased materially at early ages; consequently, where temperature differentials within the concrete are an important factor, this effect should be taken into consideration. The total heat liberated is not changed appreciably. The use of calcium chloride in warm concrete, such as may be obtained in hot weather concreting, may result in such rapid stiffening as to impede placing or finishing.

Calcium chloride generally increases expansion caused by the alkali-aggregate reaction, but the effect of calcium chloride appears to be unimportant when expansion is controlled by the use of low-alkali cement or pozzolan. It also lowers the resistance of concrete to sulfate attack. Resistance to freezing and thawing is increased at early ages by calcium chloride, but is somewhat reduced at later ages. It significantly increases the resistance of concrete to erosive and abrasive action especially at early ages.

Calcium chloride has not been found to promote corrosion of the usual reinforcement in concrete where adequate concrete cover is provided for the steel.[18] However, it should not be used where stray electric currents are expected[8,19] and should not be used in prestressed concrete because of possible stress corrosion of the prestressing steel.[20,21,22,23] Calcium chloride in concrete may be expected to aggravate corrosion of imbedded galvanized metal and of galvanized forms that are left in place. Combinations of metals, such as aluminum-alloy electrical conduit and steel reinforcing, should not be used in concrete containing calcium chloride.[24]

Calcium chloride may be especially beneficial for concrete exposed to low or freezing temperatures at early ages if used as recommended in the ACI Standard "Recommended Practice for Winter Concreting"

(ACI 604-56).[17] Calcium chloride increases the rate of early heat development and accelerates the set, but lowers the freezing point of the water in concrete only to an insignificant extent. There are no known materials that can be used to effectively lower the freezing point of water in concrete.

Difficulty may sometimes be experienced if calcium chloride is premixed with other admixtures such as water-reducing or air-entraining admixtures prior to addition to the concrete. Some combinations are compatible and can be packaged together; the components of other combinations must be added separately. The manufacturer's recommendations should be followed.

Calcium chloride may be added either dry or in solution, but the use in solution is greatly preferred to use in a solid form. When used in solution, it is convenient to prepare the solution so that 1 quart contains 1 lb of calcium chloride. When preparing the solution, care must be taken to insure proper mixing. Calcium chloride should be added to water and not water to calcium chloride, since a coating may form which is difficult to dissolve. Checks should be made on the specific gravity of the solution to be certain that proper concentration is being maintained.

Use of calcium chloride in a dry, lumpy form can result in pop-outs in concrete surfaces. However, it has been successfully used after removing lumps by means of a ¼-in. sieve. When used dry, it may be measured by volume or by weight. In using the dry material, care must be taken to insure that dry material does not become caked during storage. Storage conditions required for portland cement are adequate for calcium chloride.

Use of accelerators in concrete products

Materials which accelerate hardening and promote early strength development of concrete may prove advantageous in the manufacture of a variety of concrete products. Early attainment of strength in a building block, for example, reduces the curing period, compensates in part for slow hardening in cold weather, and decreases the time required to produce a fully matured block. Similar advantages may be obtained in the manufacture of other concrete products. During warm weather, accelerators should be used judiciously so as not to produce too rapid a set. Many plants employ only high temperature curing during summer months, and a combination of high temperature curing with an accelerator during the winter months.

WATER-REDUCING ADMIXTURES AND SET-CONTROLLING ADMIXTURES

General

Certain organic compounds or mixtures of organic and inorganic compounds are used as admixtures for both air-entrained and non-air-entrained concrete to reduce the water requirement of the mixture or to retard the set, or both. Generally, the effect of use of these materials on the hardened concrete is improved compressive strength and some improvement in impermeability and, for the types causing adequate air entrainment, improved durability under freezing and thawing conditions. A reduction in water-cement ratio increases the strength of the concrete, but the gain in compressive strength frequently is greater than is indicated by that relationship alone.

The materials that are generally available for use as water-reducing admixtures and set-controlling admixtures fall into four general classes:[3]

1. Lignosulfonic acids and their salts

2. Modifications and derivatives of lignosulfonic acids and their salts
3. Hydroxylated carboxylic acids and their salts; and
4. Modifications and derivatives of hydroxylated carboxylic acids and their salts

Admixtures of Classes 1 and 3 can be used either alone or in combination with other organic or inorganic, active or essentially inert substances. They are water-reducing, set-retarding admixtures.

Admixtures of Classes 2 and 4 are water-reducing admixtures offered as combinations of substances designed either to have no substantial effect on rate of hardening, or to achieve varying degrees of acceleration or retardation in rate of hardening of concrete; these admixtures may include an air-entraining agent.

In addition to these four main classes, compounds of other composition are sometimes used.[4] Lack of information and experience pertaining to these materials prevents specific discussion of their properties and use at this time.

Lignosulfonates are available as the calcium, sodium, or ammonium salts. Such salts may be used to extend the setting time of concrete 30 to 60 percent at temperatures of 65-100 F. In the amounts normally used, lignosulfonate retarders entrain 2 to 6 percent air in the concrete, although data are available which indicate that 6 to 10 percent has been entrained. Air-detraining admixtures may be used to reduce the air content if required (see the section on air-detraining admixtures). The composition of the portland cement affects the air-entraining properties of lignosulfonate admixtures in concrete.

Concrete containing a lignosulfonate retarder generally requires 5 to 10 percent less water than does comparable concrete without the admixture. Compressive strengths at 2 or 3 days are usually equal to or higher than those of corresponding concrete without the admixture and the strength at 28 days or later may be 10 to 20 percent higher.

Hydroxylated carboxylic acid salts act as water-reducing, non-air-entraining retarders. The rate of use is adjusted to produce the degree of retardation desired. Used in the proportion needed to retard the set by 30 percent, the water content may be reduced by 5 to 8 percent in either air-entrained or non-air-entrained concrete. The rate of bleeding and bleeding capacity are increased. Compressive strengths during the first 24 hr are lower but after 3 days are higher by 10 to 20 percent.

Increases in flexural strength of retarded concretes over those of unretarded concretes are usually obtained though they are not proportionally as great as the increases in compressive strengths.

Lignosulfonic acid salts, carboxylic acid salts, or modifications or derivatives thereof can be mixed or reacted with other chemicals that entrain air, modify setting time, or affect the strength development of concrete. Calcium chloride, neutralized wood resins, alkyl aryl sulfonates, and triethanolamine are examples of additives that have been used. The use of compounded or modified water reducers usually causes a water reduction of 5 to 10 percent at equal air content. Compressive strengths at ages greater than 2 days are usually from 10 to 20 percent higher than those of similar concretes without admixture.

The lignosulfonic materials (Classes 1 and 2) may cause some reduction in bleeding and settlement of freshly mixed concrete, depending on the degree to which they entrain air in the concrete. All four classes of material enhance the air-entraining properties of air-entraining cements as well as the amount of air entrained by a given proportion of an air-entraining admixture.

Use

Water-reducing admixtures, set-retarding admixtures, water-reducing and set-retarding admixtures, and water-reducing and accelerating admixtures should meet the applicable requirements of "Tentative Specifications for Chemical Admixtures for Concrete," ASTM C 494.

Tests should be made, if adequate information is not available, to evaluate the effect of the admixture on the properties of the concrete to be made with job materials under the anticipated ambient conditions and construction procedures. To secure meaningful results in tests of a water-reducing, set-controlling admixture, the same attention must be given to air content, bleeding, grading of the aggregate, sand content, yield, and all aspects of consistency pertinent to the work as would be required if no admixture were used. The proportion of the admixture employed is of great importance, because the proportion employed can affect simultaneously such properties as water requirement, air content, rate of hardening, bleeding, and strength of the concrete. Tests of water-reducing admixtures and set-controlling admixtures should indicate their effect on the following properties of concrete insofar as they are pertinent to the work: (1) Water requirement; (2) Air content; (3) Consistency; (4) Bleeding of water and possible loss of air from the fresh concrete; (5) Rate of hardening; (6) Compressive and flexural strength; (7) Resistance to freezing and thawing; and (8) Drying shrinkage.

Admixtures of all four classes are available in either powder or liquid form. Powders may be added with the cement or the aggregate but preferably with the latter, or, if entirely soluble, they may be dissolved in water and added as a solution. Liquids, including job-mixed solutions, may be added with the mixing water or with nonabsorptive or water-saturated aggregates, or added after the other constituents of the concrete have been partially mixed; they should not be allowed to come in contact with the cement prior to addition of the mixing water. For any given project, a fixed procedure should be adopted for control of the dispensing operation. Care should be taken to provide sufficient mixing of the concrete following addition of the admixture to assure that it is distributed uniformly throughout the batch of concrete.

Since relatively small quantities (ordinarily 1 to 13 fluid ounces or 0.2 to 1 lb per bag of cement) are used, it is important that suitable and accurately adjusted dispensing equipment be used.

Two or more admixtures of different types, such as an air-entraining admixture and a water-reducing, retarding admixture, may be used in combination in a concrete mixture in suitable proportions to achieve certain desired properties. As mentioned in the section on accelerators, however, certain combinations of admixture ingredients should not be packaged together or dispensed to the concrete through the same discharge line. The manufacturer's recommendations should be followed. Unless tests demonstrate that the several admixtures to be used simultaneously are mutually compatible when intermixed prior to their addition to the concrete, the individual admixtures should be added separately to different components of the concrete or to the concrete in the mixer during the mixing operation. Incompatibility of such admixtures when intermixed alone or in water, does not indicate that such admixtures will not be individually fully effective when combined in the concrete mixture.

Applications

Water-reducing admixtures are used to improve the quality of concrete, obtain specified strength at lower cement content, or to increase the slump of a given mixture without increase in water content. They also may improve the properties of concrete containing aggregates that are harsh, or poorly graded, or both, or may be used in concrete that must be placed under difficult conditions. They are useful when placing concrete by means of a pump, or when using a tremie.

Set-retarding admixtures are used primarily to offset the accelerating and damaging effect of high temperature, and to keep concrete workable during the entire placing period and thereby eliminate form-deflection cracks. This method is of particular value to prevent cracking of concrete beams, bridge decks, or composite construction work. Set-retarders also are used to keep concrete plastic for a sufficiently long period of time so that succeeding lifts can be placed without development of cold joints or discontinuities in the structural unit. However, their effects on rate of slump loss vary with the particular combinations of materials used.

The specific effects of water-reducing and set-controlling admixtures vary with different cements, water-cement ratios, mixing temperature, ambient temperature, and other job conditions. It is generally recommended that the proportions of the retarder used be adjusted to meet job conditions. As the concrete temperature increases, more retarder or one of a different formulation should be added to maintain a desired setting time. Adjustment of the proportions of a combination retarding, air-entraining admixture to give, at the same time, suitable retardation and air content may not be possible. Addition of increased amounts of such an admixture to maintain the required air content under adverse conditions might also cause excessive retardation and resultant delayed bleeding and finishing.

Retarders are not recommended for, and are not normally effective in, controlling false set.

GROUTING ADMIXTURES

Retarders are especially useful in cement-grout slurries, particularly if grout holes are to be redrilled, or when grouting is prolonged, or in cases where the grout must be pumped for a considerable distance, or where hot water is encountered underground.

Neat cement grouts and cement grouts containing pozzolanic materials are often used in cementing oil wells under conditions requiring that the grout remain fluid for one or more hours at elevated temperatures and pressures. In deep wells, temperatures may be up to 400 F and pressures as high as 18,000 psi. Special oil-well cements either without retarders or containing retarders introduced during the grinding operation are also available. Retarders may be added at the mixer to either a normal cement or to one that already contains a retarder, providing that in the latter case the effects of joint use of the retarding admixtures have been established.

In the cementing of oil wells and in some other grouting operations, admixtures are sometimes used to prevent the rapid loss of water from the cement paste to the surrounding formation. Some of the materials[25] suggested for this use are gels, clays, pregelatinized starch, and methyl cellulose.

Materials[25] such as bentonite clays, are used to reduce the weight of the slurries, while other materials, such as barite and iron filings, are used to increase the weight. Thickeners such as natural gums, may be added to these slurries to prevent the settling of heavy constituents.

Many other admixtures, such as gas-forming admixtures, accelerators, and others, are employed in grouting operations for special applications and are described in other sections of the report.

AIR-ENTRAINING ADMIXTURES

Effects of air entrainment

The benefits of air entrainment have been described in many articles.[26,27,28] Experience has demonstrated the superior durability of air-entrained concrete. Its use should always be required under conditions of severe natural weathering and where sodium chloride or calcium chloride is used for ice removal on pavements. Air-entrained concrete containing a large number of very small air bubbles is several-fold more resistant to frost action than non-air-entrained concrete made of the same materials. Air-entrained concrete should be a dense, impermeable mixture that is well-placed, protected, finished, and cured if maximum durability is to be obtained.

Air entrainment materially alters the properties of both the freshly mixed and the hardened concrete. Air-entrained concrete is considerably more plastic and workable than non-air-entrained concrete. It can be handled and placed with less segregation and there is less tendency for bleeding. The durability of the hardened concrete is improved by increased uniformity, decreased absorption and permeability, and by the elimination of planes of weakness at the top of lifts. These effects are due to a change in the characteristics of the concrete brought about by the presence of a large number of minute air bubbles in the paste. At a given air content, the protection afforded by the voids against damage by freezing and thawing usually is greater the larger the number of voids per unit volume of paste. This means that the voids are more effective the closer they are together.[29,30,31] The cement paste in concrete is normally protected against the effects of freezing and thawing if the spacing factor[29] of the air void system is 0.008 in. or less as determined in accordance with ASTM C 457, but for some conditions a maximum spacing factor less than 0.008 in. may be required.[32]

The air content and the size distribution of air voids produced in air-entrained concrete are influenced by many factors,[30] among the more important of which are the (1) nature and concentration of the air-entraining admixture; (2) nature and proportions of the constituents of the concrete mixture; (3) type and duration of mixing employed; (4) consistency; and (5) kind and degree of compaction applied in placing the concrete.

The use of air entrainment does not vitiate the need for control of the water-cement ratio. As the water-cement ratio is increased, the average size of the air voids, the distance between the air voids, and the freezable water content of the cement paste increase under given conditions, resulting in decreased resistance of the concrete to freezing and thawing. Resistance of concrete to laboratory freezing and thawing has not been found to be affected adversely by loss of air as a result of vibration, provided that the concrete originally contained an adequate void system. Presumably the same is true of frost resistance under field conditions.

Air entrainment, while improving both workability and durability, may reduce strength. Within the range of air content normally used, the decrease in strength usually is about proportional to the amount of air entrained. For most types of exposed concrete a slight reduction in strength is far less significant than the improved resistance to frost action. The reduction in strength will rarely exceed 15 percent in the case of compressive strength and 10 percent in the case of flexural strength. These figures are for equal cement content and with the sand and water content of the air-entrained concrete reduced to the extent permitted by the increased workability of this type of mixture.

The discussion above refers to the use of moderate amounts of entrained air, usually not more than 13 percent by volume of the mortar fraction of the concrete. In some applications of concrete, particularly in precast units, much greater quantities of entrained air are employed to produce lightweight products with superior thermal insulating properties. The latter application is discussed in the section on cellular concrete.

Air-entraining materials used as admixtures

Many materials, including natural wood resins, fats, and oils may be used in preparing air-entraining admixtures. These materials are usually insoluble in water and generally must be chemically processed before they can be used as admixtures.

Since not all such materials produce a desirable air-void system, air-entraining admixtures should meet the requirements of the ASTM Specifications for Air-Entraining Admixtures for Concrete, C 260. Conformance with these specifications will assure that the admixture functions as an air-entraining agent, that it can effect a substantial improvement in the resistance of concrete to freezing and thawing, and that none of the essential properties of the concrete (e.g., strength, volume change) are seriously impaired.

Air-entraining additions

Air-entrained concrete can also be made by using an air-entraining portland cement. Air-entraining portland cement is portland cement that contains one or more air-entraining additions (see ASTM C 219) which have been interground with it during its manufacture. Air-entraining cement should be required to meet the ASTM Specification for Air-Entraining Portland Cement, C 175.

Preparation of air-entrained concrete

At present both methods of entraining air are being used extensively and both are providing improved concrete. Adding the admixture at the mixer is to be preferred because the air content can be controlled within close limits or can be changed readily as may be indicated by the requirements of the work. Air-entrained cement may be preferred because its use is convenient, and affords some assurance of increased durability even when facilities are not available to measure the resulting air content. ACI Standard 613-54 "Recommended Practice for Selecting Proportions for Concrete,"[33] should be followed in either case.

Regardless of the method of air entrainment employed in the preparation of air-entrained concrete, the properties of the concrete-making materials, the proportioning of the concrete mixture, and all aspects of the mixing, handling, and placing procedures should be maintained as constant as feasible so that the air content of the concrete will be uniform and within the range specified for the work. The air content of the concrete should be checked and controlled during the course of the work in accordance with the recommendations of ACI Committee 611 as reported in the *ACI Manual of Concrete Inspection*.[34] Particular attention should be given to the unusually high amount of air-entraining admixture often required in concrete containing high-early-strength (Type III) portland cement, portland-pozzolan cements, fly ash, finely divided mineral admixtures such as natural pozzolans, or finely divided coloring admixtures such as untreated carbon black.

As pointed out in the sections dealing with accelerators and with water-reducing, set-retarding admixtures, some air-entraining admixtures are not compatible with other admixtures if intermixed prior to addition to the concrete and so must be added separately to the batch. The manufacturer's recommendations should be followed in such instances.

The air content, spacing factor, and other significant parameters of the air-void system in hardened concrete can be determined microscopically by several methods, the most commonly used being the linear traverse and modified point-count procedures as described in ASTM C 457.

These methods afford means to determine the air content and characteristics of the air-void system in concrete of structures.[30 (Part 4)] Use of these methods in coordination with investigations of proportioning of concrete for new projects provides greater assurance that concrete of satisfactory resistance to freezing and thawing will be obtained.[31]

Use of air-entraining admixtures in concrete products

There is no general agreement on the benefits which may accrue from the use of an air-entraining admixture in the manufacture of concrete block.[35, 36, 37] With the usual manufacturing methods, the use of an air-entraining admixture is considered by some to permit greater compaction and hence denser block and to save wear on molds. Appearance of the block may be improved, edge tear is reduced, and the block strip cleanly with sharp edges and corners. Since there is some question whether air is actually entrained in the very dry mixtures used in block manufacture, some believe that the benefits are not due to air entrainment, but rather to a surface slickness resulting from the particular type of air-entraining admixture usually used. Others have credited the use of air-entraining cement with benefits in block manufacture; while still others use a type of air-entraining admixture which probably would have no effect on the surface other than through the production of entrained air. The change in surface texture resulting from use of an air-entraining admixture, may be considered as an advantage or as a disadvantage depending on the effect being sought by the architect.

Similarly, satisfactory results using air-entraining admixtures have been reported in the manufacture of cast stone and concrete pipe. In those processes employing concrete of plastic consistency, air is no doubt entrained and such benefits as reduction of bleeding, reduction of segregation, reduction of permeability, greater resistance to the effects of freezing and thawing, and more exact reproduction of mold contours are those that would be expected in conventional concrete. Where non-plastic mixtures are used, the results are probably similar to those obtained with concrete block.

In some installations of precast concrete units, such as cribbing and curbing, there is considerable exposure to freezing and thawing action. The use of adequately prepared and controlled air-entrained concrete is the best way to improve resistance to freezing and thawing.

Mixtures made with lightweight aggregate and without entrained air are generally harsh, hence air entrainment is particularly advantageous in such concrete. Beneficial results are obtained by the addition of entrained air to both lean and rich mixtures containing any grading or type of aggregate, including lightweight aggregate. The greatest improvement is obtained, however, in harsh mixtures deficient in fines.

For best results, it is usually desirable to add an air-entraining admixture at the concrete mixer, because different amounts are necessary to produce the optimum results in various products and with use of differing methods of making the same product. The optimum amount in any particular case must be determined by experiment. Care should be exercised to avoid air contents so large that the strength of the product is greatly reduced.

Cellular concrete

Cellular concretes are those in which air or gas bubbles are substituted for all or part of the aggregate.[38] Concretes of this type are known as gas, foam, or cellular concretes.[39] Cellular concretes may be divided into two general groups: gas concrete, and foam concrete. Both are produced through the use of admixtures. Gas concrete will be discussed in a later section.

Air may be entrained in amounts from 30 to 60 percent by volume for structural cellular concrete and 70 to 85 percent for insulating cellular concrete. The air may be whipped into the mass by rapid agitation together with the addition of air-entraining admixtures such as sodium lauryl sulfate, alkyl aryl sulfonates, certain soaps, resins, or other agents. The quantities used must be much greater than those employed in the usual air-entrained concrete.

In another process the foam is produced separately using agents of the types employed to combat gasoline fires, such as air foams stabilized by hydrolyzed waste protein. In such an operation the foam is added to the cementitious slurry in the mixer in place of the conventional aggregate.

Foam concretes may vary from materials weighing as little as 20 lb per cu ft and containing no mineral aggregate, to those weighing 110 lb per cu ft or more and containing conventional aggregate. The lightest materials, possessing just sufficient strength to retain their shape in handling, are used primarily for thermal insulation. Some of the heavier ones have sufficient strength for structural applications such as in walls of dwellings.

A comparison between 40 lb per cu ft cellular concrete block made with normal-curing and similar-weight autoclaved block shows the latter having slightly better insulating properties, two to three times the strength, and 1/4 to 1/6 the volume change on drying and wetting.[40]

Such reduction of shrinkage and swelling is especially desirable in foam concrete. Shrinkage and swelling of ordinary concretes are restrained by the aggregates, whereas foam provides no restraint.

Autoclaved products with unit weights of 30 to 45 lb per cu ft have been reported with compressive strengths of from 300 to over 1000 psi, while those of 70 to 90 lb per cu ft can have strengths ranging from 3000 to 10,000 psi. The coefficient of thermal conductivity ranges from 0.85 Btu-in. per sq ft-hr-deg F for 30 lb per cu ft concrete to 3.0 to 3.5 for 90 lb per cu ft concrete.

The use of autoclaving makes feasible the addition of unusually high proportions of finely divided siliceous material to the mixture. Experimental concrete has been prepared using one part of cement to two parts of fly ash by absolute volume in combination with sufficient foam to produce a void content of 50 percent.

So-called "no-fines" concrete has been produced with air-entraining admixtures in amounts to give air entrainment of 20 to 30 percent of the volume of the concrete.[41] In effect, entrained air replaces fine aggregate. Various coarse aggregates are suitable. Such concrete can have unit weights ranging from 105 to 120 lb per cu ft and compressive strengths of from 200 to 1000 psi.

Use of air-entraining admixtures in lightweight-aggregate concrete

It is now common practice to use air-entraining admixtures in all kinds of lightweight aggregate concrete, including not only insulating and fill concrete in which such aggregates as expanded perlite and vermiculite are used, but also structural lightweight concrete containing such aggregates as expanded shale, clay, slate, or slag. Air entrainment improves workability and cohesiveness, reduces bleeding, and improves resistance to freezing and thawing. Used in optimum proportion, air entrainment increases the compressive strength of lean concrete.

Insulating concrete containing perlite or vermiculite usually contains 20 to 35 percent air. Air content of this magnitude lowers both the unit weight and the compressive strength materially. The amount of air-entraining admixture required to obtain such contents is commonly ten times as much as that needed for frost resistance in ordinary concrete. Such high air content reduces the water requirement of the concrete greatly; these mixtures are more fluid in consistency than are conventional concretes and they do not require vibration or agitation other than screeding for placement. Although slump ordinarily is not measured, optimum placeability is obtained with concretes having slumps between 5 and 8 in. Without entrained air, concretes containing vermiculite or perlite aggregates are difficult to manipulate; also the water content and unit weight are relatively high and the thermal insulation properties of the hardened concrete are not as favorable.

Due to the unfavorable shape and surface texture of the fine fraction of most lightweight aggregates used for structural concrete, it is usually desirable to use air-entraining admixtures to increase workability. Without entrained air such concrete is generally harsh, has a high bleeding rate, and high water requirement. Concretes containing more than 6 bags of cement per cu yd do not always require air entrainment for adequate workability, but in many instances even these concretes are improved in placing and finishing qualities.

Wherever exposure to severe weather is a consideration, the proportion of air-entraining admixture should be fixed at the minimum required for frost resistance. In other cases the amount should be adjusted as required for workability without excessive reduction of strength.

AIR-DETRAINING ADMIXTURES

There have been cases[42] where aggregates have released gas into, or caused excessive air entrainment, in plastic concrete which made it necessary to use an admixture able to dissipate the excess air or other gas. Also, it is sometimes desirable to remove part of the entrained air from a concrete mixture. Compounds such as tributyl phosphate, dibutyl phthalate, water-insoluble alcohols, and water-insoluble esters of carbonic and boric acids, as well as silicones, have been proposed for this purpose; however, tributyl phosphate is the most widely used material.

GAS-FORMING ADMIXTURES

Use of gas-forming admixtures to counteract settlement and bleeding

Settlement and bleeding in freshly mixed concrete are caused by gravitational settling of the individual solid particles in the mixture, sometimes accompanied by loss of water through forms or into the adjacent or underlying soil. The extent of settlement and bleeding is dependent on several factors and when excessive, or under certain concreting conditions, may result in undesirable characteristics of the hardened concrete or mortar. Accumulation of low quality matrices, laitance layers, and voids on the under side of forms, blockout cavities, reinforcing steel, or other embedded parts, or under machinery, may prevent or reduce bond, watertightness, uniformity, or strength of the concrete or mortar. This necessitates, in some instances, costly clean-up and grouting operations.

Aluminum powder[43,44] when added to mortar or concrete, reacts with the hydroxides present in fresh cement paste to produce minute bubbles of hydrogen gas throughout the matrix. Other metals such as magnesium and zinc also react with alkalies to form hydrogen, but only aluminum has received any extensive use as an admixture for concrete. The rate and extent of the reaction depends on the type and amount of aluminum powder, fineness and composition of the cement, temperature, mixture proportions, and other factors. Usually, the unpolished powder is preferred, though when a slower reaction is desired, the polished form may be advantageous. The amounts added are usually in the range of 0.005 to 0.02 percent by weight of cement, although larger amounts may be used in the production of low-strength cellular concrete.

Because of the very small quantities of aluminum powder generally used (about 1 teaspoonful per bag of cement), and because it has a tendency to float on the mixing water, it is generally premixed with fine sand, cement, or pozzolan, or incorporated in commercially available admixtures having water-reducing, set-retarding effects.

The release of hydrogen, when properly controlled, causes a slight expansion of freshly mixed concrete or mortar and thus reduces or eliminates settlement. When expansion is restrained, this will increase bond to horizontal reinforcing steel and improve the effectiveness of grout in filling fissures without excessive reduction in strength. Use of aluminum powder is particularly useful for grouting under machines or back-filling under horizontal surfaces. It is used also in grouting of post-tensioned elements in prestressed concrete. Too much aluminum powder may cause an accumulation of gas voids beneath the confining horizontal surface thus decreasing the support provided by the grout. The effect on strength depends to a considerable extent on the degree to which the tendency of the mixture to expand is restrained. Without restraint, loss of strength may be considerable, but with complete restraint, the strength is not affected appreciably and, in some cases, may even be increased slightly. Therefore, it is important that confining forms be tight and adequately closed. Use of aluminum powder or other gas-forming agents in grout or concrete will not overcome shrinkage after hardening caused by drying or carbonation.

In hot weather, hydrogen may be released too rapidly and be lost. In cold weather, chemical action may not progress fast enough to produce enough hydrogen before the mixture has set. Delayed generation of gas may or may not be harmful, depending on the rate and amount of gas generation after the concrete or grout has set and on the degree of restraint imposed. The rate of gas generation can be increased by the addition of some alkaline materials as mentioned in the next section. At normal temperatures, the aluminum reaction starts at the time of mixing and may continue for 1½ to 4 hr. At temperatures above 90 F, the reaction may be completed in 30 min and subsidence may take place until the concrete or grout takes its initial set. Approximately twice as much aluminum is required at 40 F as at 70 F to produce the same amount of expansion.

Use of gas-forming admixtures in concrete products

By using larger quantities of aluminum powder than indicated above, lightweight concretes can be produced. In some cases, alkaline compounds such as sodium hydroxide, hydrated lime, or trisodium phosphate are added to accelerate the generation of gas. If the reaction is controlled so that the gas is formed at the proper time and in appropriate amounts, the mixture increases greatly in volume. It has also been found desirable to add some type of air-entraining agent or other material such as sodium benzoate to stabilize the gas cells, thus reducing the tendency toward gas-bubble coalescence with resultant segregation and formation of layers. By varying the proportions of gas-forming admixture and carefully controlling other factors, such as temperature and the uniformity of the cement used, it is possible to produce concretes of a wide range of densities. Such concrete is made either with or without aggregates, but generally to avoid undue segregation only lightweight coarse aggregates should be used. The process is suitable not only for concrete products, but also for some cast-in-place concrete or mortar such as insulation fills where possible cracking resulting from the high potential drying shrinkage is not objectionable.

For this purpose approximately 1 lb of aluminum powder per cu yd of concrete is used, but the amount must be adjusted when any of the other ingredients or manufacturing conditions change.

Zinc and magnesium powders are also used for this purpose, while hydrogen peroxide and bleaching powder can be used in combination to produce oxygen instead of hydrogen bubbles in the concrete.

In general, the properties of gas concrete are similar to those of concrete of comparable unit weight produced with foam or with high air entrainment. For information on properties, the section of this report on cellular concrete should be consulted.

EXPANSION-PRODUCING ADMIXTURES

Usually expansion-producing materials are incorporated in "expanding" or "drying-shrinkage-compensating" cements, but since they may also be used as admixtures, a brief description is considered appropriate in this report.

Expansion-producing admixtures are materials which, during the hydration period of the concrete, either expand themselves or react with other constituents of the concrete with resulting expansion. The expansion may be of about the same magnitude as the drying shrinkage expected at later ages or it may be greater. For a given application, the extent of expansion and the time interval during which it takes place are very important and must be under control for the most satisfactory results.

For unrestrained concrete, the expansion must not take place before the concrete gains sufficient tensile strength to be stressed in tension rather than disrupted by the expanding forces. For restrained applications, the concrete must be strong enough to withstand the compressive stresses developed. It is reported that restraint in only one direction is required[45] to achieve some degree of compression in the other two orthogonal directions.

Properly timed expansion, of suitable amounts, might be employed in machinery grouting, patching, production of concrete free from shrinkage cracks, and production of self stressing, prestressed concrete. A number of expansive agents have been reported including the following:

1. Finely divided or granulated iron, and chemicals to promote oxidation of the iron, are used to produce some expanding or shrinkage-compensating mortars and concretes. The expansion is produced by the increase in solid volume as the iron is converted to iron oxides and takes place when air and moisture have access to the iron. Control of the proportion of oxidizing catalyst is required to procure the desired amount of oxidation and expansion; chlorides should not be added separately as an accelerator to the mortar or concrete. While experience has been varied, the use of mortar or concrete prepared at low water-cement ratio, and well compacted and cured, has been found adequate in many cases to prevent the continued oxidation and expansion which would otherwise occur on subsequent rewetting of the mortar or concrete. Grouts containing higher proportions of these admixtures should be employed only where confined, and the exposed surfaces should be sealed or covered by a suitable paint, sand-cement mortar, or concrete. The instructions of the manufacturer should be followed.

2. A sulfoaluminous cement for use with portland cement, made by burning a mixture of gypsum, bauxite, and limestone[46] has been manufactured in France. It is used in amounts of 9 to 25 percent by weight of the portland cement, and ground slag is also added, in amounts of 15 to 20 percent of the total. The time at which expansion takes place is controlled by varying the quantity of slag and the fineness of both the slag and the sulfoaluminous cement. Termination of expansion also has been accomplished through withholding of water for further curing. The slag is said to eventually combine with the excess calcium sulfate.

The expansion is thought to be due to the formation of hydrated calcium sulfoaluminate, the hydration reaction taking place in the solid state.

3. A Russian self-stressing cement consisting of portland cement, gypsum plaster, and aluminous cement has been reported[47,48] in which the expansion is controlled by adjustment of the hydrothermal methods of curing.

4. Recent work in the United States[45,49] has been reported on an anhydrous sulfoaluminate which added to portland cement produced degrees of expansion of a magnitude ranging from that required to give a drying-shrinkage-compensating cement to that producing a self stress in restrained concrete as high as 1200 psi. It was found that the performance of this material was affected by the characteristics of the portland cement with which it is used, the water-cement ratio, and the curing conditions.

5. The production of self-stressing concretes utilizing the periclase in the cement (or of that added to it) has been reported.[50] Hydration of the magnesia is accelerated and controlled by controlling the steam treatment.

FINELY DIVIDED MINERAL ADMIXTURES

General

Finely divided mineral admixtures may be classified into three types: Those which are relatively inert chemically, those which are pozzolanic, and those which are cementitious. Many of the materials so used are powders as fine as or finer than portland cement. They therefore serve to influence the physical properties of the fresh paste in much the same manner as does cement and may be used to augment the cement in mixtures deficient in fine materials. Since the distinction between the fine material in the cement and that in the aggregates is more or less arbitrary, such mineral admixtures may also serve as correctives for deficiencies in aggregate gradation. Many concrete mixtures, in order to have the necessary workability and plasticity, must contain a larger amount of portland cement than would be required to develop adequate strength. A portion or all of this additional cement may frequently be left out when the mixture is proportioned with a suitable mineral admixture. In such application, the chemical characteristics of the admixture are of secondary importance.

Finely divided materials which are either pozzolanic or cementitious contribute to the strength development of the concrete, and mixtures in which they are used usually require considerably less cement to produce a given strength. In addition to the changes in the hardened concrete which may result from the modification of the physical properties of the freshly mixed paste through the use of finely divided admixtures, the pozzolanic and cementitious materials modify further the physical and chemical properties of the final product.

Since the finely divided materials discussed in this section, regardless of whether they are chemically inactive, pozzolanic, or cementitious, when used in concrete, are neither aggregate nor portland cement, they are by definition *admixtures* when they are added to the concrete batch as separate ingredients, either before or during mixing. Such materials are, by definition, *additions* when interground or blended with portland cement. As noted above, a concrete mixture containing such a material, if properly and economically proportioned, will usually include a smaller proportion of portland cement than would otherwise be required. There has, therefore, been a tendency to refer to these materials as "replacements" or "substitutes" for part of the portland cement. Some concrete mixtures having characteristics deficient in some respects can be improved by adding a finely divided mineral admixture as an additional ingredient without altering the relative proportions of the other ingredients.

Types of finely divided materials

1. *Relatively chemically inert materials*

This class includes such materials as ground quartz, ground limestone, bentonite, hydrated lime, and talc.

2. *Cementitious materials*

The cementitious materials include natural cements, hydraulic limes, slag cements (mixtures of blast-furnace slag and lime), and granulated iron blast-furnace slag.

3. *Pozzolans*

Pozzolan is defined in ASTM C 219 as "a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties." Fly ash, volcanic glass, diatomaceous earths, and some shales or clays, either heat-treated or raw, are examples of pozzolanic materials. A summary of information on the significance of tests and properties of these materials has been given by Meissner.[51]

4. *Specifications*

Specifications for finely divided mineral admixtures include the following:

(a) *Natural cement* — ASTM C 10; Federal Specification SS-C-185a.*
(b) *Hydraulic lime* — ASTM C 141.
(c) *Slag cement* — ASTM C 358; Federal Specification, SS-C-218a.*
(d) *Raw or calcined natural pozzolans* — ASTM C 402.
(e) *Fly ash* — ASTM C-350. This specification covers the use of fly ash either as a pozzolan or as an admixture where the use of increased quantities of suitable fine material is indicated to promote workability and plasticity, or as an admixture for both of these purposes.

Effects on properties of freshly mixed concrete

In mixtures deficient in "fines" (particularly material passing the No. 200 sieve) addition of a finely divided mineral admixture improves workability, reduces the rate and amount of bleeding, and increases the strength. In general, the higher the specific surface of the admixture, the smaller the volume required to produce a given effect on workability. When an appropriate quantity of a mineral admixture is used, no increase in total water content of the concrete is required and drying shrinkage and absorptivity of the hardened concrete are not much affected.

Addition of a mineral powder to mixtures not deficient in fines, particularly mixtures rich in portland cement, generally decreases workability for a given water content. For that reason, the addition of mineral powders to such mixtures without a reduction in cement generally entails an increase in the total water content of the concrete, and may result in an increase in drying shrinkage and absorptivity, and a decrease in strength.

Of the various factors determining the bleeding characteristics and the degree of plasticity of fresh concrete, the amount of surface area of the solids per unit of water volume is the most important.[52] In a con-

*Obtainable from Business Service Center, Federal Supply Service, General Services Administration, Regional Office Building, Washington, D.C. 20407.

crete mixture in which the ratio of surface area of solids to volume of water is low, the paste is thin and watery. Consequently, the aggregate particles are only slightly separated by layers of paste and the mixture lacks plasticity and tends to segregate.[53] A low ratio of surface area to water volume denotes also excessive particle interference according to Weymouth's criterion.[54]

When the ratio of surface area of solids to volume of water is low, the rate of bleeding is relatively high. Moreover, most of the bleeding does not appear at the surface; that is, the aggregate particles settle for a short period until they establish point-to-point contacts that prevent further settlement. The watery paste continues to bleed within the pockets defined by aggregate particles leaving layers of water at the undersides of the particles. Thus, with mixtures having the characteristics just discussed, bleeding tends to reduce homogeneity of the concrete. In extreme cases, the lack of homogeneity is manifested by open fissures under the aggregate particles large enough to be visible in a cross section of the concrete.

These undesirable effects can be ameliorated by increasing the ratio of surface area of solids to volume of water in the paste. This generally increases the stiffness of the paste and at a given slump effects a wider separation of the aggregate particles in the concrete. The ratio of surface area of solids to volume of water may be increased by increasing the amount of cement (lowering the water-cement ratio of the paste) or by adding a suitable mineral admixture.

Although under some circumstances the addition of mineral powders lowers the amount of water and air voids in concrete, such admixtures should not be regarded as void-fillers—at least not in the sense of providing small particles to fill the voids between the larger particles. A mixture of cement and water (or of cement, mineral admixture, and water) forms a soft, plastic paste which envelops the aggregate particles and, in the course of the mixing process, separates them. Thus, in a sense, a function of the mineral admixture is to increase the paste content of the mixture, and thereby its capacity for plastic deformation.

It is desirable to select an admixture having favorable physical characteristics. The admixture should be such that the paste will contain a maximum proportion of solid matter and a minimum proportion of water. This requires that the mineral particles do not have unfavorable shapes, and that the surface area be not too high.

Whether or not the paste in a given concrete mixture can be benefited by means of a mineral admixture depends on various factors. One criterion is whether the solid content of the mixture can thereby be increased or, in other words, the sum of water and air contents be reduced. To ascertain whether this criterion can be met by using a given mineral admixture, it is necessary to know the interrelationship between water-plus-air content, water-cement ratio, and paste content for the specific cement and aggregate to be used. A discussion of these relationships as affected by the use of mineral admixtures was given on pp. 139-143 of the 1954 report.[2]

Proportioning

Finely divided mineral admixtures have been used in a wide range of proportions of admixture to portland cement.

In some cases, either the weight or the solid volume of the admixture used in the reproportioned mixture will be equal to that of the portland cement that it replaces. Usually, however, the volume of admixture employed will be larger, if the concrete containing the admixture is proportioned for optimum properties and maximum economy. These materials should therefore properly be regarded as another class of ingredients of concrete and not as cement replacements.

Some materials, especially those of greatest fineness, usually are employed in proportions of 5 to 15 percent of the amount of cement; others usually in the range of 15 to 30 percent; and, in some cases, the amount of a pozzolanic or cementitious admixture used is greater than the amount of portland cement. Recommendations have been made[55] for the use of the pozzolan, fly ash, in structural concrete in which mixtures without pozzolan containing 4, 5, or 6 bags of portland cement per cu yd may be reproportioned to leave out 94, 83, and 71 lb of portland cement, respectively, when 150 to 175, 125 to 137, and 100 lb of fly ash are used, to produce concrete of equivalent strength and workability. These recommendations also contemplate that, in general, about 2 gal. less water will be required per cu yd in the mixtures of equal workability and strength reproportioned with fly ash than was required in comparable mixtures without fly ash. Reduction of water content of concrete mixtures containing fly ash is attributed to the beneficial effect on the workability of the concrete produced by the typically spherical particle shape and smooth, dense surface texture of fly ash particles. Many pozzolanic materials, due to the size, shape, surface texture, and grading of their particles, cause an increase in water requirement of concrete in which they are used as compared to that of comparable concrete without pozzolan. When water content is increased, the absorption, drying shrinkage, and permeability of the concrete may be increased.

In the Trief process, which has been used in heavy construction in Europe, granulated blast-furnace slag is ground with water and added to the concrete mixture as a slurry. For the Bort Dam, which contains 863,000 cu yd of concrete, the cementitious medium consisted of 68.5 percent wet ground slag, 30 percent portland cement, and 1.5 percent sodium chloride.

Some indication of suitable proportions for use of cementitious and pozzolanic admixtures in concrete is given by specifications for blended cements. ASTM and Federal specifications for portland blast-furnace slag cement require the use of from 25 to 65 percent slag as an interground addition in the finished product. Federal specifications for portland-pozzolan cement require that the percentage of pozzolan by weight be between 15 and 35 percent; the ASTM specifications require between 15 and 50 percent. Materials listed in the specifications that can be used as pozzolanic additions in the manufacture of portland-pozzolan cement include "clays, shales, diatomaceous earths, tuffs, volcanic ash, and pumicite, either calcined or uncalcined, and fly ash."

Effects on strength

The effect of a mineral admixture upon the strength of concrete varies markedly with the properties of the particular admixture used and with the characteristics of the concrete mixture in which used. Generally, the strength of lean mixtures is increased and the strength of rich mixtures is decreased. Contributions to strength by pozzolanic and cementitious admixtures usually are relatively slow, particularly at low temperatures. Under favorable curing conditions, strengths at later ages typically will be higher than is obtained with portland cement alone, although large variations result with the use of different admixtures. In all cases, moist curing must be continued longer for proportionate development of potential strength than is necessary with concrete not containing a pozzolanic or cementitious admixture.

Effects on sulfate resistance

Pozzolanic and cementitious materials have been used in structures exposed to sea water or other sulfate-bearing water.[56] Pozzolanic materials generally are employed in the proportion of from 1 part of pozzolan to 5 parts of portland cement to 1 part of pozzolan to 2 parts of portland cement, calculated either by weight or by absolute volume. They generally have a lower specific gravity than does portland cement; therefore, if used on a weight basis, a greater total absolute volume of cementitious material results. Use of pozzolanic material with other than sulfate-resisting portland cements generally increases resistance of the concrete to aggressive attack of sea water, sulfate-bearing soil solutions, and natural acid waters. The relative improvement is greater for concrete of low cement content. The use of pozzolan with sulfate-resisting portland cements does not increase sulfate resistance and, if chemically active aluminum compounds are present in the pozzolan, may actually cause a reduction in sulfate resistance of the concrete.[57,58]

Effects on temperature rise

Pozzolanic and cementitious admixtures have been used in large hydraulic structures where it is desirable to reduce the portland cement content in order to decrease temperature rise resulting from heat liberation on hydration of the cement. The proportions used for this purpose usually are similar to those used for improving sulfate resistance.

Effects on expansion caused by alkali-aggregate reaction

The use of pozzolan for the specific purpose of preventing excessive expansion caused by alkali-aggregate reaction was recommended in 1947.[59] However, there have been only a few instances in which an admixture has been used in concrete containing known reactive aggregates and a known high-alkali cement and the pozzolan relied on to prevent the expected excessive expansion. Performance data on these projects are not available to the committee.

The alkali-aggregate reaction involves the interaction of alkalies in portland cement with certain siliceous constituents of the aggregates in concrete. Products of the reaction can cause excessive expansion, cracking, and general deterioration of the concrete.[60-67] The term "alkalies" refers to the sodium and potassium present in relatively small proportions expressed as sodium oxide (sum of the percentage of Na_2O and 0.658 times the percentage of K_2O). When this particular type of distress of concrete was first described by Stanton in 1940[68,69] the only apparent remedies were the use of portland cement of low-alkali content (0.60 percent or less computed as Na_2O) or the avoidance of reactive aggregates.

To date, field experience and service records of concrete in which cements of low-alkali contents and reactive aggregates were employed have not indicated that excessive expansions will occur. However, a considerable number of laboratory tests have indicated that excessive expansions are possible under some exposure conditions if some types of aggregate are combined with low-alkali cement.[59,70] It has also been indicated by laboratory tests that certain natural or artificial pozzolans are capable of reducing the expansion caused by alkali-aggregate reaction; however, some other pozzolans have shown little ability to prevent excessive expansion. It is, therefore, necessary to evaluate by test the ability of individual pozzolanic materials to control alkali-aggregate reaction. An accelerated test to determine reduction effected in expansion of mortar is usually used for this purpose.[71]

Three classes of pozzolans include the materials which have been found to reduce significantly the expansion caused by alkali-aggregate reaction in concrete or mortar: (1) "amorphous" siliceous or siliceous and aluminous substances, including some opals and highly opaline rocks, certain volcanic glasses, diatomaceous earth, calcined clays of the kaolinite type, and some fly ashes; (2) clays of the montmorillonite type, containing calcium as the exchangeable cation, which have been calcined in the 1000 to 1800 F range, but not at a temperature sufficiently high to destroy the crystalline structure; (3) combinations of the above two categories including siliceous shales; e.g., certain siliceous shales and certain altered pumicites which are mixtures of montmorillonite clay and volcanic glass.[72] The amount of suitable pozzolan required in a concrete to control this reaction will vary with individual aggregates and with the alkali content of the cement. Ample protection should generally be obtained by use of proportions ranging from 20 to 35 percent by weight of the cement. The use of these amounts usually does not impair physical properties of the concrete such as strength and workability. Certain materials, however, when finely divided and of high opal content (e.g., certain diatomaceous earths and opaline cherts) will prevent expansion when used in amounts of less than 15 percent by weight of the cement. In proportions of 10 percent or less by weight of the cement, certain pozzolans may increase expansion of concrete containing reactive aggregate and high-alkali cement, presumably because interaction of a portion of the cement alkalies with the pozzolan produces a ratio of reactive silica to the available alkalies which more closely approaches the optimum (or "pessimum") for formation of expansive alkali-silica gel. Consideration must also be given in the use of finely divided materials to the usually attendant increase in water requirement. See also the section of this report on chemical admixtures to reduce alkali-aggregate expansion.

DAMP-PROOFING AND PERMEABILITY-REDUCING ADMIXTURES

Some concrete dams, retaining walls, tanks, and other structures show evidence of leakage. Usually, such leakage is the result of faulty production and placement of concrete, or it is due to cracks in the structure. When properly proportioned concrete mixtures are used and placed with high-class workmanship under qualified inspection, the concrete in a structure should be virtually impermeable, although leakage may still occur through cracks. The use of certain admixtures has been advocated as a means of correcting deficiencies of the mixture, or of facilitating better workmanship. Such use of admixtures is for the purpose of beneficially modifying the characteristics of fresh concrete. An admixture described as a damp-proofer, or as a permeability-reducing agent, may have some such beneficial effect on the properties of fresh concrete not directly indicated by the name. For example, it may promote entrainment of air and thus may properly be considered an air-entraining admixture. In such a case, not only this section but also the section of this report on air-entraining admixtures is pertinent. In this section we deal with those aspects directly implied by the terms damp-proofing or permeability-reducing. Such terms imply an effect on the properties of hardened concrete, apart from whatever effect the admixture might have on freshly mixed concrete. The ensuing discussion therefore deals with the possible effects of such agents on the properties of hardened concrete.

The terms "damp-proofing" and "water-proofing" imply prevention of water penetration of dry concrete, or stoppage of transmission of water through unsaturated concrete. However, admixtures have not been found to produce such effects; the terms have come to mean a reduction in rate of penetration of water into dry concrete, or in rate of transmission of water through unsaturated concrete from the damp side to the dryer side.

The term *permeability* usually refers to a coefficient giving the rate at which water is transmitted through a saturated specimen of concrete, under an externally maintained hydraulic gradient.

Admixtures for "damp-proofing" or "water-proofing"

Admixtures for damp-proofing include soaps, butyl stearate, and certain petroleum products.[73-78] The soaps comprise salts of fatty acids, usually calcium or ammonium stearate or oleate. With the exception of butyl stearate, they cause entrainment of air during mixing. Among petroleum products are mineral oils, asphalt emulsions, and certain cut-back asphalts.

Admixtures such as these, by reducing penetration of the visible pores, may retard penetration of rain into concrete block made of nonplastic mixtures. Test data show that they reduce also the rate of penetration of moisture into the micropores of dry concrete, but there is no indication that there are comparable effects on the transmission of moisture *through* unsaturated concrete, except when the concrete contains paste having relatively high porosity. A paste of high porosity results from low cement content and correspondingly high water-cement ratio, lack of curing, or from both factors. If the concrete has a sufficiently low porosity such as that obtained by producing a well-cured paste having a water-cement ratio not over 0.6 by weight, damp-proofing agents give no appreciable improvement.

The Building Research Advisory Board,[79] reported that in the opinion of the majority of 61 interrogated observers, damp-proofing admixtures are not "... effective or acceptable in controlling moisture migration through slabs-on-ground." It also reported that a special advisory committee to the Building Research Advisory Board reached the following conclusion on the basis of data from tests on moisture transmission *through* unsaturated concrete slabs: "The Committee does not find adequate data to demonstrate the effectiveness of any admixture to reduce the transmission of moisture through concrete slabs-on-ground in a manner sufficient to replace either a vapor barrier or granular base, or both, under conditions where such protection would be needed."

Permeability-reducing admixtures

Admixtures of the kinds discussed above do not reduce the coefficient of permeability of saturated concrete. However, mineral powders, properly proportioned, reduce the permeability of mixtures in which the cement content of the paste is relatively low. Under conditions where this effect is obtained, there is usually also a reduction in the amount of water per cubic yard, and thus a small reduction in porosity.

The reduction of total water content by means of a water-reducing admixture should reduce the total porosity slightly, but there are no adequate data to demonstrate that permeability is thereby reduced materially.

Accelerating admixtures such as calcium chloride increase the average rate of hydration and thereby reduce the length of time required for a concrete mixture to attain a given fraction of its ultimate degree of impermeability. However, any advantage attained this way is likely to be temporary since, if conditions are such that water is being transmitted through the concrete, they are also conducive to continued hydration of cement.

BONDING ADMIXTURES

Bonding admixtures are water emulsions of any of several organic materials that are mixed with portland cement or mortar grout for application to an old concrete surface just prior to placing topping or patching mortar or concrete, or are mixed with the topping or patching material. Their function is to increase the bond strength between the old and new concrete or, through modification of the properties of the new concrete, to reduce the bond stresses developed, or both. This procedure is used in patching of eroded or spalled concrete or to add relatively thin layers of resurfacing. These materials have been found useful also in the formulation of cement paints and for bonding of portland cement plaster.

The commonly used bonding admixtures are made from natural rubber, synthetic rubber, or any of a great number of organic polymers or copolymers. The polymers include polyvinyl chloride, polyvinyl acetate, acrylics, and butadiene-styrene copolymer.

Bonding admixtures fall into two general categories, namely, re-emulsifiable types and non-re-emulsifiable types. The non-re-emulsifiable types are resistant to water and are therefore better suited to exterior application and use in areas where moisture is prevalent. Ability of a bonding admixture to cure and harden in contact with moist concrete, and the retention of cured strength in the presence of water, are important features to be considered in selection of materials for this use.

These emulsions are generally added to the mixture in proportions equivalent to 5 to 20 percent by weight of the cement, the actual quantity used depending on the type of bonding mixture being prepared and the job conditions. Bonding admixtures usually cause entrainment of air and a sticky consistency in grout mixtures. They are effective only on clean, sound surfaces since the strength of the bond is only as good as the strength of the material to which it is attached.

CHEMICAL ADMIXTURES TO REDUCE ALKALI-AGGREGATE EXPANSION

Test data indicate that small additions of certain chemical substances may be effective in decreasing expansion resulting from alkali-aggregate reaction.[80, 81] Outstanding reductions in expansion of laboratory mortar specimens have been reported for additions of 1 percent by weight of the cement of lithium salts and for additions of about 2 to 7 percent of certain barium salts. Moderately reduced expansions were also obtained with certain protein air-entraining admixtures and with some water-reducing, set-retarding admixtures. It was found that some of these substances were more effective in reducing expansion than others. The results reported are limited and further work is needed. There is some evidence that expansions due to alkali-aggregate reaction are slightly lowered by air entrainment. The use of pozzolans to prevent excessive expansion caused by alkali-aggregate reaction is described in the section on finely divided mineral admixtures.

CORROSION INHIBITING ADMIXTURES

Many investigators have studied the corrosion of iron and steel with particular reference to protective coatings. Cushman and Gardener[82] state, "the consensus . . . appears to be that concrete furnishes ample protection to the steel embedded in it, except in certain cases in which infiltrating or percolating waters find a way through the concrete, washing away the free alkali present in the form of lime or calcium hydroxide." According to Cushman,[83] the slightly soluble chromates should theoretically be the best protectives against the corrosion of iron and steel. However, in studying paints prepared with chrome pigments, he found that pigments which contained any soluble impurities that tended to stimulate corrosion did not provide protection for the metal. He concluded that, "if the surface of iron is subjected to the action of two contending influences, one tending to stimulate corrosion and the other to inhibit it, the result will be a breaking down of the defensive action of the inhibitor at the weakest points, thus localizing the action and leading to pitting effects."

The problem of corrosion of reinforcing steel in concrete has generally been limited to concrete exposed to saline or brackish waters or soils containing chlorides from which chlorides can reach the steel either by diffusion through the concrete or by entrance through cracks. Probably because it was recognized that concrete itself was a good protective coating, and because the work with paints indicated that inhibitors such as chromates would not provide protection under conditions where chlorides could enter the concrete, there is little information in the technical literature pertaining to the use of corrosion-inhibiting admixtures in concrete.

Lewis, Mason and Brereton[84] reported on work done with reference to the process patented by Dougill[85] for the North Thames Gas Board in which sodium benzoate is used as a corrosion-inhibiting admixture to protect the steel in reinforced concrete. In this process 2 percent sodium benzoate is used in the mixing water; or a 10 percent benzoate cement slurry is used to paint the reinforcement, or both. Sodium benzoate is also an accelerator of compressive strength. It was found by chemical analysis that the sodium benzoate remained present as such in the concrete after 5 years exposure, 1.3 percent having been recovered from concrete to which 2.0 percent had been added.

Kondo, Takeda and Hideshima[19] studied the corrosion of steel in concrete exposed either to alternating or direct current and found very little corrosion in concrete containing no admixture and severe corrosion in concrete that contained chloride. They found that an admixture containing calcium lignosulfonate decreased the rate of corrosion of the steel in the concrete that contained calcium chloride.

In the manufacture of certain concrete products containing steel, it might be desirable to accelerate the rate of strength development by use of both a chemical accelerator and heat, the latter usually in the form of steam at atmospheric pressure. When calcium chloride is used as the accelerator in this type of curing, the rate of corrosion of the steel has been found in laboratory studies to be accelerated. However, Arber and Vivian[10] found that certain compounds which contain an oxidizable ion, such as stannous chloride, ferrous chloride, and sodium thiosulfate, act as accelerators as does calcium chloride, but also appear to cause less corrosion that the latter. Stannous chloride appeared to be the best of the products tried and 2 percent of the salt by weight of cement was more effective than 1 percent, and as effective as greater amounts, both from the standpoint of acceleration and resistance to corrosion. Stannous chloride oxidizes to stannic chloride when in solution and may also oxidize in situ where lean relatively permeable concrete furnishes inadequate protection against access to oxygen. For effective use, the salt must be added to the concrete in the stannous form and a dense concrete must be used.

Sodium nitrite has been investigated by Moskvin and Alekseyev[86] as an inhibitor of corrosion of steel in autoclaved products. These authors suggest that the high alkalinity, which is normally present in concrete and which serves to passivate the steel, may be considerably reduced by autoclave treatment especially when siliceous admixtures are present. Two to three percent sodium nitrite by weight of cement was found to be an efficient inhibitor under these conditions. Sarapin[87] found by storage tests that 2 percent sodium nitrite was effective in preventing corrosion of steel in concrete containing calcium chloride.

Warnings have been sounded against the use of inhibitors. For example, the South African National Building Research Institute[88] made the following statement, "*Integral Additives:* Although certain inert and reactive materials have shown promise, their use cannot be recommended at this stage, as insufficient evidence of their effectiveness or possible disadvantages is available." Evans[89] made the following statement, "a beneficial effect from the inhibitor addition might reasonably be expected if the steel surface is clean and chlorides absent, but under such conditions there is unlikely to be serious trouble even without added inhibitor. Where rust particles occur there is a risk that the rust, preventing the inhibitor from reaching the metal below it, may establish the combination of small anodes and large cathodes.... making matters worse."

FUNGICIDAL, GERMICIDAL, AND INSECTICIDAL ADMIXTURES

It has been suggested that certain materials may either be ground into the cement or added as admixtures to impart fungicidal, germicidal, or insecticidal properties to hardened cement pastes, mortars, and concretes. These materials include polyhalogenated phenols,[90,91] dieldren emulsion,[92] and copper compounds.[93,94]

FLOCCULATING ADMIXTURES

The addition to cement pastes, mortars, and concretes of certain synthetic polyelectrolytes are reported to increase the bleeding rate and decrease the bleeding capacity of cement pastes, to reduce flow of pastes and mortars, and to increase the green strength and cohesiveness of mortars.[95,96]

COLORING ADMIXTURES

Pigments are often added to produce color in the finished concrete. The requirements of suitable coloring admixtures include: (1) color fastness when exposed to sunlight; (2) chemical stability in the presence of alkalinity produced in the set cement; (3) no adverse effect on setting time or strength development of the concrete; and (4) stability of color in autoclaved concrete products during exposures to the conditions in the autoclave. For information on color fastness, resistance to alkali, and stability at elevated temperatures of various pigments reference may be made to Payne.[97]

Pigments frequently used are shown in Table 1.

TABLE 1—COLORS OF VARIOUS PIGMENTS

Shades of color	Pigment
Grays to black	Black iron oxide Mineral black Carbon black
Blue	Ultramarine blue Phthalocyanine blue
Bright red to deep red	Red iron oxide
Brown	Brown iron oxide Raw and burnt umber
Ivory, cream, or buff	Yellow iron oxide
Green	Chromium oxide Phthalocyanine green
White	Titanium dioxide

Inorganic pigments are usually added in amounts of 2 to 10 percent by weight of the cement, and should not be added in amounts greater than 10 percent except as noted below. Most of these pigments are available either as a natural or earth color, or as a synthetic material. Although the unit price may be higher, the synthetic materials are often more economical in use because of better color values brought about by greater fineness and purity. They may also be more uniform than the natural materials.

The pigments should preferably be thoroughly mixed or interground with the dry cement, but they also are used successfully when blended into the dry concrete mixture before addition of the mixing water.

Trial mixtures, with observation of the dry concrete resulting, should be made to determine the required dosage of pigment. The color produced in concrete varies also with the tools and procedures employed in finishing of surfaces and with conditions of curing.

Carbon black is added in amounts of ½ to 1 percent by weight of cement. The addition of carbon black usually requires a considerable increase over the normal amount of air-entraining agent required to produce the desired air content in concrete.

White cement rather than gray is often more effective and economical than a white pigment such as titanium dioxide. For light, clear colors the use of clean, light-colored, fine aggregates as well as white cement is required.

The earlier organic phthalocyanine greens and blues were not satisfactory for use in concrete, but materials are now available, either dispersed in water, or as dry pigments, which are satisfactory.

Some pigments break down during autoclave curing, most of the iron oxide yellows, oranges, and browns dehydrating to reds, and the ferrous oxide portion of some black iron oxides oxidizing to rust-colored ferric oxide during cooling. In autoclaved products, pigments containing silica, such as the umbers and mineral black, may be added in amounts greater than 10 percent by weight of cement, because they may cause a compensating gain in strength through pozzolanic action.

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Guide for Structural Lightweight Aggregate Concrete

Reported by ACI Committee 213

This guide summarizes the present state of the technology. It presents and interprets the data on lightweight aggregate concretes from many laboratory studies, accumulated experience resulting from greatly increased and successful use, and performance of structural lightweight aggregate concrete in service.

The guide is intended for the architect, engineer, contractor, concrete producer, and student. It includes a definition of lightweight aggregate concrete for structural purposes; it discusses in condensed fashion the production methods and inherent properties of lightweight aggregates for structural concrete. This is followed by current practices on proportioning, mixing, transporting, placing; properties of hardened concretes; and finally, the design of structural concrete, with special reference to the 1977 ACI Building Code, 318-77.

Keywords: abrasion resistance; air-entrained concretes; air entrainment; bond (concrete to reinforcement); cement content; **coarse aggregates;** compressive strength; **concrete durability;** creep properties; curing; deflection; fine aggregates; fire resistance; fire tests; flexural strength; fly ash; freeze-thaw durability; fresh concretes; hardened concretes; **lightweight aggregate concretes; lightweight aggregates; lightweight concretes;** mechanical properties; mix proportioning; modulus of elasticity; physical properties; production methods; quality control; ready-mixed concrete; shear strength; shrinkage; splitting tensile strength; structural design; tensile strength; thermal conductivity; thermal expansion; thermal properties; thermal transmittance; water-cement ratio; workability.

Foreword

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Structural lightweight aggregate concrete has come of age as an important and versatile material in modern construction. It has many and varied applications: multistory building frames and floors, curtain walls, shell roofs, folded plates, bridges, prestressed or precast elements of all types, and others. In many cases the architectural expression of form combined with functional design can be achieved more readily in structural lightweight concrete than in any other medium. Many architects, engineers, and contractors recognize the inherent economies and concomitant advantages offered by this material, as evidenced by the many impressive lightweight concrete structures found today throughout the world. Structural lightweight aggregate concrete is structural concrete in the strictest sense.

Since the development of structural lightweight concrete has been essentially parallel to the earlier development of normal weight concrete, considerable use has been made of the large amount of information available on normal weight concrete. However, when the unique characteristics of lightweight aggregate and concrete have required departures from customary practice, these have been detailed in this Guide.

Because structural lightweight aggregate concrete is the newer material, engineering research laboratories and the lightweight aggregate and concrete industries have had to develop a large amount of information on physical and structural properties in a short time. It was necessary to learn the qualities and behavior of the material while it was being used. This has produced the unusual circumstance that data for such properties as creep, shrinkage, and modulus of elasticity are frequently more accurately known for a structural concrete made with a given lightweight aggregate than for concrete made with a given normal weight aggregate.

This guide supersedes the original committee report published in the Aug. 1967 ACI JOURNAL.
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Chapter 1 — Introduction

1.1 — Objective of the guide

The objective of the Guide for Structural Lightweight Aggregate Concrete is to recommend the best practices of preparing and applying structural lightweight aggregate concrete. Using such practices, structures may be designed and their performance predicted with the same high degree of accuracy, and with the customary factors of safety, that is attained for normal weight structural concrete.

1.2 — Historical development

1.2.1 *Early development through World War II* — Prior to 1917, S. J. Hayde developed a rotary kiln process for heat-expansion of shales and clays to form hard, lightweight material which served as aggregates in making concrete of substantial strength and low weight. At about the same time, F. J. Straub pioneered in the use of bituminous coal cinders as an aggregate for manufacture of concrete masonry units which attained high production volume following World War I, and which are still being manufactured today. Commercial production of expanded slag began in 1928; and in 1948, the first structural quality sintered shale lightweight aggregate was produced using a coal-bearing shale in eastern Pennsylvania.

One of the earliest uses of reinforced lightweight concrete was in the construction of ships and barges by the Emergency Fleet Building Corp. of World War I.[1] Concrete of the required compressive strength of 5000 psi (34.47 MPa) was obtained with a unit weight of 110 pcf (1760 kg/m^3) or less, using expanded shale aggregate. The Park Plaza Hotel in St. Louis and the Southwestern Bell Telephone Building in Kansas City, built during the 1920's are other examples of early applications of reinforced lightweight concrete in buildings. In the early 1930's, the use of lightweight concrete for the upper roadway of the San Francisco-Oakland Bay Bridge was a key to the economical design of the bridge. During World War II, history repeated itself with the construction of 105 lightweight concrete ships,[2] thereby conserving steel plate for other essential uses.

1.2.2 *Post World War II development* — Considerable impetus was given to the development of lightweight concrete shortly after World War II when a National Housing Agency survey was conducted on the potential of lightweight concrete for home construction. This led to an extensive study of concretes made with lightweight aggregates. Sponsored by the Housing and Home Finance Agency,[3] parallel studies were conducted simultaneously in the laboratories of the National Bureau of Standards[4] and the U.S. Bureau of Reclamation[5] to determine properties of concrete made with a broad range of lightweight aggregate types. These studies, and the earlier work by Richart and Jensen,[6] and Washa and Wendt,[7] and others, focused attention on the structural potential of some lightweight aggregate concrete and initiated a renewed interest in lighter weight for building frames, bridge decks, and precast products in the early 1950's.

The addition of four stories to an existing department store in Cleveland was made possible by the reduced dead load of lightweight concrete without necessity of foundation modification. Similarly, following the collapse of the original Tacoma Narrows Bridge, it was replaced by another suspension structure incorporating additional roadway lanes without the necessity of replacing the original piers, due to the use of structural lightweight concrete in the deck.

During the 1950's many multistory structures were designed from the foundations up to take advantage of reduced dead weight with lightweight concrete. Examples are the 42-story Prudential Life Building in Chicago, which incorporated lightweight concrete floors, and the 18-story Statler Hilton Hotel in Dallas, which was designed with a lightweight concrete frame and flat plate floors.

Structural applications such as these stimulated more concentrated research into the properties of lightweight concrete by several important national and international organizations. Similarly construction of aggregate plants was accelerated, until today lightweight aggregates of structural quality are available in most parts of the United States and Canada and many other countries. Development of knowledge and construction of major structures in nearly all metropolitan areas of the United States and Canada continued in the 1960's at an increasing tempo.

At the end of 1978 there were approximately 39 rotary kiln expanded shale plants, 4 sintering process expanded shale plants, 11 expanded blast furnace slag plants, and 1 pelletized or extruded fly ash sintering plant in the United States and Canada.

1.3 — Economy of structural lightweight concrete

The use of lightweight aggregate concrete in a structure is usually predicated on a lower overall cost of the structure. While lightweight concrete may cost more per cu yd than normal weight concrete, the structure may cost less as a result of reduced dead weight and lower foundation costs. This is the basic reason, in most cases, for using structural lightweight concrete. Economy then depends on attaining a proper balance among cost of concrete per volume, unit weight, and structural properties. Normal weight concrete may be the least in cost per cu yd, but will be heavier, resulting in greater dead loads, increased sizes in many sections, and therefore may require more concrete and reinforcing steel. Concrete in which the aggregate is entirely lightweight will usually be the most expensive per cu yd, but will be the lightest, resulting in reduced dead loads, lower volume of concrete and reinforcing steel, and lower handling and forming costs. Lightweight concrete in which natural sand is used for part or all of the fine aggregate will lie between the two extremes of cost of concrete per cu yd and dead weight.

1.4 — Lightweight aggregates — classifications

There are many types of aggregates available which are classed as lightweight, and their properties cover wide ranges. To delineate those types which can be classed as structural, and which are therefore pertinent to this Guide, reference is made to a concrete "spectrum," Fig. 1.4. This diagram indicates the approximate 28-day, air-dry unit weight range of three types of lightweight aggregate concretes along with the use to which each type is generally associated. The indicated dividing weights of these types (and the end points of each bar for each of the aggregates) are approximate only and should not be considered precise.

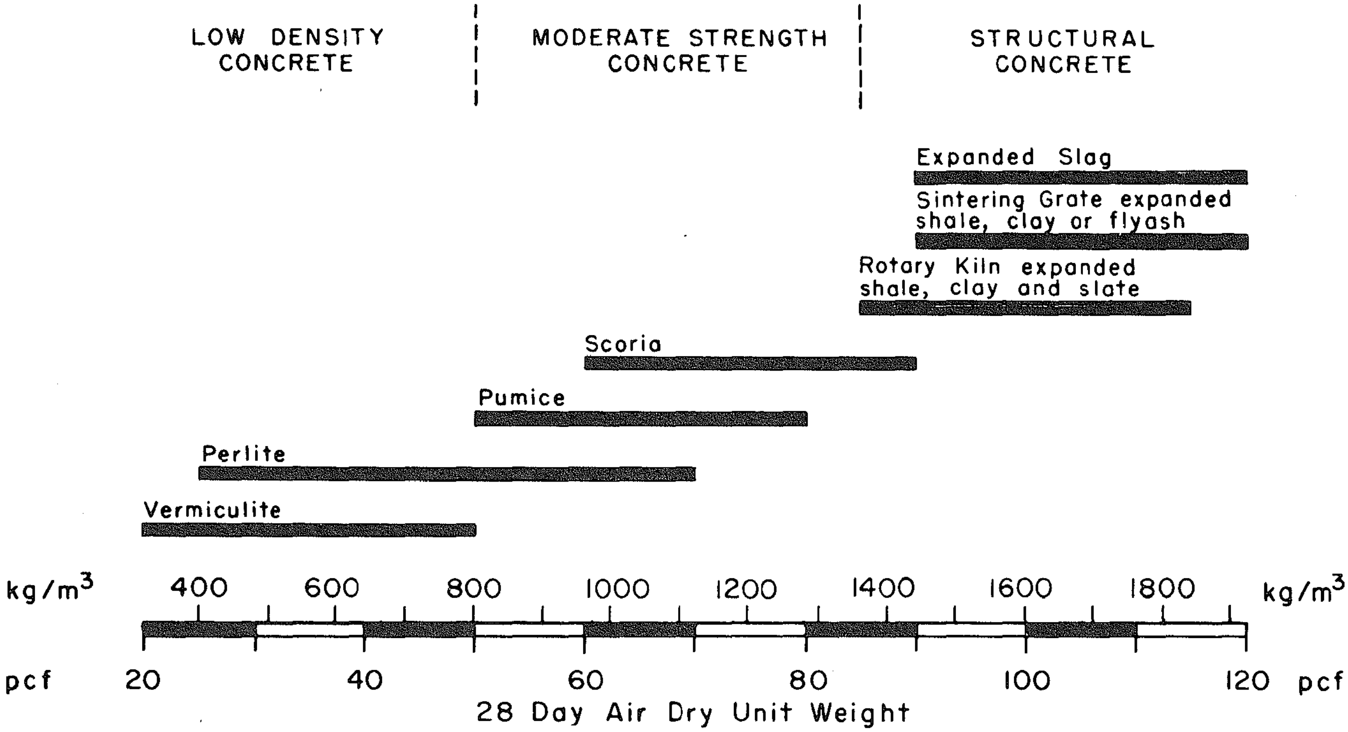


Fig. 1.4 — Approximate unit weight and use classification of lightweight aggregate concretes

1.4.1 *Low density concretes* — These especially light concretes are employed chiefly for insulation purposes. With low unit weights, seldom exceeding 50 pcf (800 kg/m³), heat insulation values are high. Compressive strengths are low, ranging from about 100 to 1000 psi (.69 to 6.89 MPa).

1.4.2 *Structural concretes* — Concretes with full structural efficiency contain aggregates which fall on the other end of the scale and which are generally made with expanded shales, clays, slates, slags, and pelletized fly ash. Minimum compressive strength, by definition, is 2500 psi (17.24 MPa) (see Section 1.5). Most structural lightweight aggregates are capable of producing concretes with compressive strengths in excess of 5000 psi (34.47 MPa) and, with a number of these, concretes can be made with strengths considerably greater than 6000 psi (41.36 MPa). Since the unit weights of structural lightweight aggregate concretes are considerably greater than those of low density concretes, insulation efficiency is lower. However, thermal insulation values for structural lightweight concrete are substantially better than for normal weight concrete.

1.4.3 *Moderate strength concretes* — The use of these concretes requires a fair degree of compressive strength, and thus they fall about midway between the structural and low density concretes. These are sometimes designated as "fill" concretes. Compressive strengths are approximately 1000 to 2500 psi (6.89 to 17.24 MPa) and insulation values are intermediate.

1.5 Definition of structural lightweight aggregate concrete

For clarification of the intent of this Guide, the following definition of structural lightweight aggregate concrete has been established:

> "Structural lightweight aggregate concretes are defined as concretes having a 28-day compressive strength in excess of 2500 psi (17.24 MPa) and a 28-day, air-dry unit weight not exceeding 115 pcf (1850 kg/m³)."

It should be understood that this definition is not a specification. Job specifications may, at times, allow unit weights up to 120 pcf (1900 kg/m³). Although structural concrete with an air-dry unit weight of 90 to 100 pcf (1450 to 1660 kg/m³) is often used, most lightweight concrete structures weigh between 100 and 110 pcf (1600 to 1760 kg/m³). The aggregate producers in various localities should be contacted prior to design for advice on the range of unit weights available.

1.6 — Structural lightweight aggregates

1.6.1 *Processed aggregates* — This Guide presents a summary of existing knowledge of elastic properties, compressive and tensile strength, time-dependent properties, durability, fire resistance, and other properties of structural lightweight aggregate concrete. It also recognizes that satisfactory field performance records are more important than results of laboratory studies. Laboratory data and field experience are available to satisfy these criteria mainly with respect to processed aggregates meeting the requirements of ASTM C330[8a] i.e., rotary kiln expanded shales, clays, and slates; sintered shales, clays, and pelletized or extruded fly ash; and expanded slags.

1.6.2 *Naturally occurring and unprocessed aggregates* — It is recognized that structural concrete may be made with other types of lightweight aggregates, for example, with naturally occurring scoria and pumice, and with suitable cinders. Lack of detailed laboratory and field information on structural properties and applications of concrete made with these materials has precluded their inclusion in this Guide. If, however, information is provided in specific cases to indicate properties and performance equivalent to the types considered herein, the guidance provided may be considered to apply.

1.6.3 *Definition of terms* — For simplicity, the term "shale," as used in many portions of this Guide, applies equally to aggregates processed from shales, clays, or slates. Further, the terms "structural lightweight concrete" and "structural lightweight aggregate concrete", used interchangeably in this Guide, should be interpreted as indicating structural concrete containing structural lightweight aggregate. The term "all-lightweight" indicates concrete in which both the coarse and fine fractions are lightweight aggregates; the term "sand-lightweight" indicates concrete with coarse lightweight aggregate and in which all of the fine fraction is natural sand. In many instances only partial replacement of the lightweight fines with natural sand is employed, and this will be so indicated in this Guide.

Chapter 2 — Structural lightweight aggregates

2.1 — Scope

A knowledge of the characteristics of the lightweight aggregate used is of prime importance to the designer and user of structural lightweight concrete. In this chapter general information is given on the types of lightweight aggregates commonly used in structural concrete, their method of production, and their basic properties.

2.2 — Definitions

2.2.1 *Fine lightweight aggregates* — These size fractions of aggregates are composed primarily of processed or naturally occurring cellular materials of mineral origin which (a) are suited to the production of structural lightweight concrete as defined in Sections 1.5 and 2.2.3; (b) are properly graded with 85 to 100 percent passing the No. 4 sieve [3/16 in. (5 mm)]; (c) have a dry, loose weight exceeding 70 pcf (1120 kg/m^3); and (d) comply with all other requirements of ASTM C330.[8a]

2.2.2 *Coarse lightweight aggregates* — The larger size fractions of lightweight aggregates are composed primarily of processed or naturally occurring cellular materials of mineral origin which (a) are suited to the production of structural lightweight concrete as defined in Sections 1.5 and 2.2.3; (b) are properly graded from 100 percent passing a designated maximum size sieve; (c) have a dry, loose weight not exceeding 55 pcf (880 kg/m^3); and (d) comply with all other requirements of ASTM C330.[8a] One or more of the following gradations are generally available:

- Structural coarse, 3/4 in. to No. 4 (19mm to 5mm) or 1/2 in. to No. 4 (13mm to 5mm)
- Medium coarse, 3/8 in. to No. 8 (10 mm to 2 1/2 mm)

2.2.3 *Structural lightweight aggregate concrete* — As previously defined (Section 1.5), such concrete: (a) has a minimum compressive strength at 28 days of 2500 psi (17.24 MPa); (b) has a corresponding air-dry unit weight not exceeding 115 pcf (1850 kg/m^3); and (c) consists entirely of lightweight aggregates or a combination of lightweight and normal weight aggregates.

2.3 — Internal structure of aggregates

In all cases the lightweight aggregates used in structural concrete are light in weight due to the cellular structure of the individual aggregate particles. This cellular structure within the particles is formed at high temperatures, generally 2000 F (1100 C) or higher, by one or more of the following processes:

(a) Formation of gases, due to reaction of heat on certain constituents in the raw materials, coincidental with incipient fusion of the mineral, so that the gases are entrapped in a viscous, pyroplastic mass causing bloating or expansion.

(b) Subjecting a softened or molten mass to intermixing with controlled amounts of water or steam so that a cellular structure is produced by entrapped steam and other gas and is retained on cooling of the mass.

(c) Burning off of combustible materials in a matrix.

The cells in the aggregate particle may vary from microscopic to macroscopic in size, and may be predominently interconnected or discrete.

2.4 — Production of aggregates

Raw materials used in commercial production of structural lightweight aggregates are generally (a) suitable natural deposits of shales, clays, or slates; (b) by-products of other industries, such as iron blast furnace slags or fly ash from the burning of powdered coal in thermoelectric power plants. The raw materials may require from little to extensive preparation prior to treatment to produce expansion. In many cases crushing to suitable sizes is the only prerequisite. In the cases of finely divided materials such as silty and laminar clays, and fly ash, the raw material may need to be agglomerated with water, or possibly require addition of supplementary binder, fuel, gas-forming or fluxing agents, prior to heating.

Several different methods are used to produce structural lightweight aggregates, and the aggregates produced may vary widely in their characteristics. No single description will apply fully to any raw material or process. A generalized description follows for the several principal processes used.

2.4.1 *Rotary kiln process* — Basically the rotary kiln is a long, nearly horizontal cylinder lined with refractory materials. Raw material is introduced in a continuous stream at the upper end, and due to slow rotation and slope of the kiln, it progresses to the lower or burner end. The heat causes simultaneous formation of gases and onset of a pyroplastic condition in the material. The viscosity of the softened mass is sufficient to entrap the gases and to form an internal cellular structure. This structure is retained on cooling as a vitrified hard material.

2.4.1.1 *Crushed material.* In one variation of the rotary kiln process, the bloated material is discharged, cooled, and then crushed and screened to required aggregate gradations. The resultant particles tend to be cubical or angular in shape and to have a varying percentage of particles with a smooth shell.

2.4.1.2 *Presized or "coated" material.* In another variation, raw material is presized, by crushing and screening or by pelletizing, before introduction into the kiln and the individual particles are bloated with little or no agglomeration. The resultant particles tend to have a smooth shell or coating over the cellular interior.

2.4.1.3 *Combination material.* Frequently there is a combination of the two procedures in which most of the coarse aggregate will consist of uncrushed particles, obtained by screening, and most of the fine particles are obtained by crushing the fired product.

2.4.2 *Sintering process* — In the sintering process, raw materials are used which either contain carbonaceous matter that serves as fuel or are mixed with fuel, such as finely ground coal or coke.

2.4.2.1 *Crushed material.* In one variation of this process an even layer of such a mixture, suitably premoistened, is carried by a traveling grate under drying and ignition hoods and subsequent burners in such a manner that burning, initiated at the surface, continues through the full depth of the bed. Gases are formed causing expansive action, coincident with the onset of pyroplasticity, so that the material is sufficiently viscous to entrap the gas and thereby create the cellular structure. The clinker formed is then cooled, crushed, and screened to required aggregate gradations. In some cases the cellular structure results from the burnout of carbonaceous matter and loss of moisture, and fusion of fine particles of the original raw material. The finished product tends to be generally sharp and angular with a vesicular surface texture.

2.4.2.2 *Pelletized material.* In a second variation of the sintering process, clay, pulverized shale or fly ash is mixed with moisture and fuel, and then pelletized or extruded before burning. The resultant product tends to be generally rounded or cylindrical in shape.

2.4.3 *Expansion of slag* — Three main processes are used in expanding molten blast furnace slag.

2.4.3.1 *Machine process.* The molten slag at a temperature in excess of 2200 F (1200 C) is rapidly agitated in a machine with a controlled amount of water and subsequently cooled and crushed. The cellular structure is formed primarily by entrapment of steam, and secondarily from gases evolved by reaction of minor constituents in the slag with the water vapor.

2.4.3.2 *Water jet process.* The molten slag, at temperatures in the range of 2200 to 2500 F (1200 to 1400 C), is treated with a controlled amount of water forced into the mass in jets under high pressure and is subsequently cooled and crushed. Expansive action occurs as entrapped water turns to steam and causes formation of the cellular structure.

2.4.3.3 *Pelletizing process.* The molten slag at a temperature in excess of 2200 F (1200 C) is treated with limited amounts of water and distributed by a vibrating, water cooled, carbon feeder to a rotating drum. Fins on the drum break the slag into small particles that solidify into rounded pellets as they are thrown through the air.

2.5 — Aggregate properties

Each of the properties of lightweight aggregates may have some bearing on the properties of the plastic and hardened concrete. It should be recognized, however, that properties of lightweight concrete, in common with those of normal weight concrete, are greatly influenced by the quality of the cement paste. Specific properties of aggregates which may affect the properties of the concrete are as follows:

2.5.1 *Particle shape and surface texture* — Lightweight aggregates from different sources or produced by different methods may differ considerably in particle shape and texture. Shape may be cubical and reasonably regular, essentially rounded, or angular and irregular. Surface textures may range from relatively smooth with small exposed pores to irregular with small to large exposed pores. Particle shape and surface texture of both fine and coarse aggregate influence proportioning of mixes in such factors as workability, fine-to-coarse aggregate ratio, cement content, and water requirement. These effects are analogous to those obtained with normal weight aggregates of such diverse particle shapes as exhibited by rounded gravel, crushed limestone or traprock, or manufactured sand.

2.5.2 *Bulk specific gravity* — Due to their cellular structure, the specific gravity of lightweight aggregates is lower than that of normal weight aggregates. The bulk specific gravity of lightweight aggregate also varies with particle size, being highest for the fine particles and lowest for the coarse particles, with the magnitude of the differences depending on the processing methods. The practical range of bulk specific gravities of coarse lightweight aggregates, corrected to the dry condition, is about 1/3 to 2/3 of that for normal weight aggregates. For specific gravities below this range the cement requirement may be uneconomically high to produce the required strength, and above this range the weight may be too high to meet ASTM requirements for lightweight concrete.

With present ASTM test methods, it may be difficult to accurately determine bulk specific gravity and water absorption for some coarse lightweight aggregates and for many fine lightweight aggregates. ACI Committee 211,[9a,9b] however, uses the concept of a "specific gravity factor" which serves in lieu of the bulk specific gravity (see Section 3.4.3).

2.5.3 *Unit weight* — Unit weight of lightweight aggregate is significantly lower, due to the cellular structure, than that of normal weight aggregates. For the same gradation and particle shape, unit weight of aggregate is essentially proportional to specific gravity. However, aggregates of the same specific gravity may have markedly different unit weights, because of different percentages of voids in the dry-loose, or dry-rodded volumes of aggregates of different particle shapes. The situation is analogous to that of rounded gravel and crushed stone which, for the same specific gravity and grading, may differ by 10 pcf (160 kg/m^3) in the dry, rodded condition. Rounded and angular lightweight aggregates of the same specific gravity may differ by 5 pcf (80 kg/m^3) or more in the dry, loose condition, but the same weight of either will occupy the same volume in concrete. This should be considered in assessing the concrete-making properties of different aggregates.

2.5.4 *Maximum size* — The maximum size grading designations of lightweight aggregates generally available are 3/4 in. (19 mm), 1/2 in. (13 mm), or 3/8 in. (10 mm). Maximum size of aggregate influences such factors as workability, ratio of fine to coarse aggregate, cement content, optimum air content, potential strength ceiling, and drying shrinkage. When comparisons are made between lightweight concrete and normal weight concrete with respect to such factors, the comparison should be made on the basis of the same maximum size.

2.5.5 *Strength of lightweight aggregates* — The strength of aggregate particles varies with type and source and is measurable only in a qualitative way. Some particles may be strong and hard, and others weak and friable. There is no reliable correlation between aggregate strength and concrete strength and lack of particle strength may not preclude use of an aggregate in structural concrete.

2.5.5.1 *Strength ceiling.* The concept of "strength ceiling" may be useful in indicating the maximum compressive strength attainable in concrete made with a given aggregate using a reasonable quantity of cement. A mix is near its strength ceiling when similar mixes containing the same aggregates and with higher cement contents have only slightly higher strengths. It is the point of diminishing returns, beyond which an increase in cement content does not produce a commensurate increase in strength. The strength ceiling for some lightweight aggregates may be quite high, approaching that of high quality normal weight aggregates.

Strength ceiling is influenced predominantly by the coarse aggregate. It has been found that the strength ceiling can be increased appreciably by reducing the maximum size of the coarse aggregate for most lightweight aggregates. This effect is more apparent for the weaker and more friable aggregates. In one case, the strength attained in the laboratory for concrete containing 3/4 in. (19 mm) maximum size of a specific lightweight aggregate was 5000 psi (34.47 MPa); for the same cement content [750 lb per cu yd (450 kg/m^3)] the strength was increased to 6100 and 7600 psi (42.06 MPa and 52.4 MPa) when the maximum size of the aggregate was reduced to 1/2 in. (13 mm) and 3/8 in. (10 mm), respectively. Concrete unit weights were concurrently increased by 3 and 5 pcf (48 and 80 kg/m^3).

2.5.6 *Moisture content and absorption* — Lightweight aggregates, due to their cellular structure, are capable of absorbing more water than normal weight aggregates. Based on a 24 hr absorption test, lightweight aggregates generally absorb from 5 to 20 percent by weight of dry aggregate, depending on the pore structure of the aggregate. Normally, however, under conditions of outdoor storage in stockpiles, moisture content will usually not exceed two-thirds of the 24 hr absorption.

By contrast, normal weight aggregates usually will absorb less than 2 percent of moisture. However, the moisture content in a normal weight aggregate stockpile may be as high as 5 to 10 percent or more. The important difference is that the moisture content in lightweight aggregates is largely absorbed into the interior of the particles whereas in normal weight aggregates it is largely surface moisture. These differences become important in mix proportioning, batching and control as discussed in Sections 3.4, 3.5, and 3.7.

Rate of absorption in lightweight aggregates is a factor which also has a bearing on mix proportioning, handling, and control of concrete, and depends on the aggregate particle surface pore characteristics plus other factors. It should be noted that the water which is internally absorbed in the aggregate is not immediately available to the cement as mixing water, as will be discussed in Section 3.2.3. Nearly all moisture in the natural sand, on the other hand, may be surface moisture which is available to the cement.

Chapter 3 — Proportioning, mixing and handling

3.1 — Scope

Proportioning of structural lightweight concrete mixtures is the determination of economical combinations of the several constituents — portland cement, aggregate, water, and usually admixtures — in a way that the optimum combination of properties is developed in both the plastic and hardened state.

A prerequisite to the selection of mixture proportions is a knowledge of the properties of the constituent materials. Generally these constituents are required to comply with the pertinent ASTM specification.

Based on a knowledge of the properties of the constituents, and their interrelated effects on the concrete, structural lightweight concrete can be proportioned and produced to have, within reasonable limits, the specific properties most suited to the finished structure.

It is within the scope of this chapter to discuss:

(a) Criteria on which concrete mixture proportions are based

(b) The materials which make up the concrete mixture

(c) The methods by which these are proportioned.

The subjects of mixing, delivery, placing, finishing, and curing also will be discussed, particularly where these procedures differ from those associated with normal weight concrete. The chapter will conclude with a brief discussion on laboratory and field control.

3.2 — Mix proportioning criteria

Chapter 4 indicates a broad range of values for many physical properties of lightweight concrete. Specific values depend on the properties of the particular aggregates being used and on other conditions. In proportioning a lightweight concrete mix, the engineer is concerned with obtaining predictable specific values of properties for a particular situation.

The specifications of the structural engineer, for lightweight concrete, usually require minimum permissible values for compressive strength, maximum values for slump, and both minimum and maximum values for air content. For lightweight concrete, a limitation is always placed on the maximum value for unit weight.

Insofar as physical properties of the concrete are concerned, the usual specification is limited to these items. From a construction standpoint, such properties of freshly mixed concrete as bleeding, workability, and finishability must also be considered. It is possible in mix proportioning, especially with lightweight concrete, to optimize these properties. Some properties are to a large extent interdependent and improvement in one property, such as workability, may affect other properties such as unit weight or strength. The final criterion to be met is over-all performance in the structure as intended by the architect/engineer.

3.2.1 *Specified physical properties*

3.2.1.1 *Compressive strength.* This property is also discussed in Section 4.3. The various types of lightweight aggregates available will not all produce similar compressive strengths for concretes of a given cement content and slump.

Compressive strength of structural concrete is specified according to engineering requirements of a structure, not according to the ability of one or another of available aggregates used in concrete to provide that strength. Normally, strengths specified will range from 3000 to 4000 psi (20.68 to 27.58 MPa) and less frequently up to 6000 psi (41.36 MPa) or higher. It should not be expected that the higher strength values can be attained consistently by concretes made with every lightweight aggregate classified as "structural," although some are capable of producing very high strengths consistently.

3.2.1.2 *Unit weight.* From the load-resisting considerations of structural members, reduced unit weight of lightweight concrete can lead to improved economy of structures despite an increased unit cost of concrete.

Unit weight is therefore a most important consideration in the proportioning of lightweight concrete mixtures. While this property depends primarily on the unit weight or density of the lightweight or normal weight aggregates, it is also influenced by the cement, water and air contents, and to a small extent, by the proportions of coarse to fine aggregate. Within somewhat greater limits the unit weight can be varied by adjusting proportions of lightweight and normal weight aggregates. For instance, if the cement content is increased to provide additional compressive strength, the unit weight of the concrete will be increased only about 3 pcf (48 kg/m^3). On the other hand, complete replacement of the lightweight fines with natural sand will increase the unit weight by 10 pcf (160 kg/m^3) or more at the same strength level. This should also be considered in the over-all economy of structural lightweight concrete.

If the concrete producer has available several different sources of lightweight aggregate, optimum balance of cost, and performance of concrete may require detailed investigation. Only by comparing concretes of the same compressive strength and of the same air-dry unit weight can the fundamental differences of concretes made with different aggregates be properly evaluated.

In some areas, only a single source of lightweight aggregate is available. In this case, the concrete producer needs only to determine that weight level of concrete which satisfies the economy and specified physical properties of the structure.

3.2.1.3 *Modulus of elasticity.* This property is discussed in detail in Sections 4.6 and 5.3. Although values for E_c are not always specified, information is usually available for concretes made with specific lightweight aggregates.

3.2.1.4 *Slump.* Slump should be the lowest value consistent with the ability to satisfactorily place, consolidate, and finish the concrete. (See Section 3.6.1 on finishing.)

3.2.1.5 *Entrained air content.* Air entrainment in lightweight concrete, as in normal weight concrete, improves durability. Moreover in concretes made with some lightweight aggregates, it is a particularly effective means of improving workability of otherwise harsh mixtures. The mixing water requirement is then lowered while maintaining the same slump, thereby reducing bleeding and segregation.

Recommended ranges of total air contents for lightweight concrete are:

Maximum size of aggregate	Air content percent by volume
3/4 in. (19 mm)	4 to 8
3/8 in. (10 mm)	5 to 9

At times there is a temptation to use a large proportion of natural sand in lightweight concrete to reduce costs, and then to use a high air content to meet weight requirements. Such a practice usually becomes self-defeating because compressive strength is thereby lowered 150 psi (1.03 MPa) or more for each increment of one percent of air beyond the recommended ranges. The cement content must then be increased to meet strength requirements. Although the percentages of entrained air required for workability and frost resistance reduce the unit weight of the concrete, it is not recommended that air contents be increased beyond the upper limits given above, simply to meet unit weight requirements. Adjustment of proportions of aggregates, principally by limiting the normal weight aggregate constituent, is the safest, and usually the more economical way to meet specified unit weight requirements.

3.2.2 *Workability and finishability*

3.2.2.1 *Workability.* Workability is probably the most important property of freshly mixed lightweight concrete. Without adequate workability it is difficult, if not impossible, to attain all the other desired properties of hardened concrete. The most satisfactory method developed to evaluate this property is the slump test when used in conjunction with the judgment of the technician.

The engineer should also keep in mind that lightweight concrete with entrained air has an established record of durability, and that the percentages of entrained air required for workability will usually also be sufficient to impart durability and other desirable properties.

3.2.2.2 *Finishability.* With some lightweight aggregates a properly proportioned, cohesive, lightweight concrete mixture with good workability will normally be finishable. Other lightweight aggregates may be deficient in minus No. 30 (0.6 mm) sieve material. When this occurs, the finishability can usually be improved by using a portion of natural sand, by increasing the cement content, or by using satisfactory mineral fines. If practical, sands with a low fineness modulus, such as those used in masonry mortars or finer, should be selected to supplement such lightweight fines. With increased fineness, less sand will be required to provide satisfactory finishability; thus the increase in weight of concrete will be smaller.

3.2.3 *Water-cement ratio* — With lightweight concrete, the water-cement ratio is not generally used, primarily due to uncertainty of calculating that portion of the total water in the mix which is applicable to the water-cement ratio. The water absorbed in the aggregate prior to mixing is not included as part of the cement paste, and complication is introduced by absorption of some indeterminate part of the water added at the mixer. However, it is quite probable that this absorbed water is available for continued hydration of the cement after normal curing has ceased. The general practice with lightweight aggregates is to proportion the mix, and to assess probable physical characteristics of the concrete, on the basis of a given cement content at a given slump for particular aggregates.

3.3 — Materials

Concrete is composed essentially of cement, aggregates and water. In some cases an admixture is added, generally for the purpose of entraining air, but occasionally for special reasons such as modifying setting time or reducing water content. When ingredients vary, as in the case of aggregates from different sources, cements of different types, or by the use of admixtures, concrete properties may differ appreciably even though the cement content and slump are held constant. It is desirable, therefore, to make laboratory tests of all the ingredients, and to proportion concrete mixtures to meet specifications and specific job requirements with the actual combinations of materials that are economically available.

3.3.1 *Hydraulic cement* — The cement should meet the requirements of ASTM C150[10a] (portland cement) or ASTM C595[10b] (blended cements). Where close control of air content is required, the use of air-entraining agents rather than air-entraining cement is preferable since the amount of entrained air depends on characteristics of the fine aggregates and on the mixing conditions. Section 4.4 discusses cement content and its influence on the properties of concrete.

3.3.2 *Lightweight aggregates* — Lightweight aggregate should meet requirements of ASTM C330[8a] for lightweight aggregates for structural concrete. Surfaces of aggregate particles have pores varying in size from microscopic to those visible to the eye. Water absorption and rate of absorption may vary widely. These differing characteristics account for the wide range in amounts of mixing water needed to produce a concrete of a given consistency with different aggregates. This wide range in water requirements is reflected in a corresponding range of cement contents necessary to produce a given strength with aggregates from different sources. The inherent strength of different aggregates also has an important effect on the cement requirement, particularly for higher strength concretes. The recommendations of lightweight aggregate producers generally provide the best estimate of the cement content and other mix proportions that should be used as a starting point in trial batches for selecting mix proportions.

3.3.3 *Normal weight aggregates* — Normal weight aggregates used in structural lightweight concrete should conform to the provisions of ASTM C33.[8b] If finer sand is desired as a supplement, it should conform to ASTM C144.[8c]

3.3.4 *Admixtures* — Admixtures should conform to appropriate ASTM specifications, and excellent guidance for use of admixtures may be obtained from the ACI Committee 212 report, "Admixtures for Concrete".[11]

3.4 — Proportioning and adjusting mixes

Proportions for concrete should be selected to make the most economical use of available materials to produce concrete of the required physical properties. Basic relationships have been established which provide guides in approaching optimum combinations of materials, but final proportions should be established by laboratory trial mixes, which are then adjusted to provide practical field batches.

The principles and procedures for proportioning normal weight concrete, such as the absolute volume method described below may be applied in many cases to lightweight concrete. With some aggregates, these procedures are difficult to use, and other methods have been developed. The local aggregate producers should be consulted for the particular recommended procedures.

3.4.1 *Absolute volume method* — In utilizing the absolute volume method, the volume of plastic concrete produced by any combination of materials is considered equal to the sum of the absolute volumes of cement, aggregate, net water, and entrained air. Proportioning by this method requires the determination of water absorption and the bulk specific gravity of the separate sizes of aggregates in a saturated surface-dry condition. The principle involved is that the "mortar" volume consists of the total of the volumes of cement, fine aggregate, net water, and entrained (or entrapped) air. This mortar volume must be sufficient to fill the voids in a volume of dry, rodded coarse aggregate, plus sufficient additional volume to provide satisfactory workability. This recommended practice is set forth in ACI 211.1-70,[9b] and it represents the most widely used method of proportioning for normal weight concrete mixtures. While the saturated surface-dry condition is most fine and many coarse lightweight aggregates[13,14] may be difficult to assess accurately, the absolute volume method can be useful in selecting proportions for structural lightweight concretes with some lightweight aggregates.

3.4.2 *Volumetric method* — The volumetric method is described with examples by ACI Committee 211.[9b] It consists essentially of making a trial mix using estimated volumes of cement, coarse and fine aggregate, and sufficient added water to produce the required slump. The resultant mix is observed for workability and finishability characteristics. Tests are made for slump, air content, and fresh unit weight. Calculations are made for yield (the total batch weight divided by the plastic unit weight) and for actual quantities or weights of materials per unit volume (cu yd or m^3) of concrete. Necessary adjustments are calculated and further trial mixes made until satisfactory proportions are attained. Prerequisite to the trial mixes is a knowledge of the dry-loose unit weights of aggregates, the moisture contents of the aggregates, an approximation of the optimum ratio of coarse and fine aggregates, and an estimate of required cement content to give the strength desired.

3.4.3 *Specific gravity factor method — Trial mix basis* — The specific gravity factor method, trial mix basis, is described with examples in ACI 211.2-69. A trial batch is prepared as in Section 3.4.2 and observations and tests made as mentioned. Displaced volumes are calculated for the cement, air, and total water (added water plus absorbed water). The remaining volume is then assigned to the coarse and fine aggregates, assuming that the volume occupied by each is proportional to its dry-loose unit weight. The specific gravity factor is calculated as the relationship between the dry weight of the aggregate in the mix and the displaced volume it is assumed to occupy. The value so determined is not an actual specific gravity but is only a factor. This factor may, however, be used in subsequent calculations as though it were the apparent specific gravity, using the principles of absolute volumes, so long as the moisture content and density of the aggregates remains unchanged.

3.5 — Mixing and delivery

The fundamental principles of ASTM C94[8d] apply to structural light-weight concrete as they do to normal weight concrete. Also, it is recommended that immediately prior to discharge, the mixer should be rotated approximately ten revolutions at mixing speed to minimize segregation.

In those cases involving aggregates with relatively low water absorption, no special prewetting is required prior to batching and mixing of the concrete. Such aggregates are sometimes stocked in the kiln-dry condition, and at other times they contain some amount of moisture. These aggregates may be handled according to the procedures which have been established in the ready-mixed concrete industry.[15] In so treating these aggregates, it should be realized that the water to be added at the batching plant should provide the required slump at the job; i.e., the added water may give high slump at the plant but water absorption into the aggregate will provide the specified slump at the building site.

In other cases, the absorptive nature of the lightweight aggregate may require prewetting to as uniform a moisture content as possible, or premixing with water, prior to addition of the other ingredients of the concrete. The proportioned volume of the concrete is then maintained and slump loss during transport is minimized.

3.6 — Placing

There is little or no difference in the techniques required for placing lightweight concrete from those utilized in properly placing normal weight concrete. ACI 304[16] discussed in detail proper and improper methods of placing concrete. The most important consideration in handling and placing concrete is to avoid separation of the coarse aggregate from the mortar portion of the mixture. The basic principles required to secure a good lightweight concrete job are:

- A workable mix utilizing a minimum water content
- Equipment capable of expeditiously handling and placing this concrete
- Proper consolidation
- Good quality workmanship

A well proportioned lightweight concrete mix can generally be placed, screeded, and floated with substantially less effort than that required for normal weight concrete. Over-vibration or over-working is often a principle cause of finishing problems in lightweight concrete. Such abnormal practice only serves to drive the heavier mortar away from the surface where it is required for finishing, and to bring an excess of the lighter coarse aggregate to the surface. "Floating" of coarse light-weight aggregate can also occur in mixes in which the slump exceeds the recommendations of Section 3.6.1.1.

3.6.1 *Finishing* — Good floor surfaces are achieved with properly proportioned quality materials, skilled supervision, and good workmanship. The quality of the job will be in direct proportion to the efforts expended to assure that all of the above essentials are maintained throughout the construction. Proper finishing of lightweight concrete floors is described by ACI Committee 302[17] and in the Expanded Shale, Clay and Slate Institute's Information Sheet No. 7.[18]

3.6.1.1 *Slump.* Slump is a most important factor in achieving a good floor surface with lightweight concrete and generally should be limited to a maximum of 4 in. (10 cm). A lower slump, of about 3 in. (8 cm), imparts sufficient workability and also maintains cohesiveness and "body", thereby preventing the lighter coarse particles from working up through the mortar to the surface. (This is the reverse of normal weight concretes where segregation results in an excess of mortar at the surface.) In addition to "surface" segregation, a slump in excess of 4 in. (10 cm) will cause unnecessary finishing delays.

3.6.1.2 *Surface preparation.* Surface preparation prior to troweling is best accomplished with magnesium or aluminum screeds and floats which minimize surface tearing and pullouts. Vibrating screeds and "jitterbugs" (grate tamper or roller type) may be used to advantage in depressing coarse particles and developing a good mortar surface for troweling.

3.6.1.3 *Good practice.* A good finish on lightweight concrete floors can be obtained as follows:

(a) Prevent segregation by:
   1. Providing a well-proportioned and cohesive mix
   2. Keeping the slump as low as possible
   3. Avoiding over-vibration

(b) Time the finishing operations properly

(c) Use magnesium, aluminum, or other satisfactory finishing tools

(d) Perform all finishing operations after free surface bleeding water has disappeared

(e) Cure the concrete properly

3.6.2 *Curing* — On completion of the final finishing operation, curing of the concrete should begin as soon as possible. Ultimate performance of the concrete will be influenced by the extent of curing provided. The two references[17,18] of Section 3.6.1 contain excellent information on proper curing of concrete floor slabs. The two methods of curing commonly used in the field are (a) water curing (wet coverings, ponding and sprinkling or soaking), and (b) moisture retention cure (polyethylene film, waterproof paper, and spray-applied curing compound membranes). In construction practice, 7 days of curing is generally considered adequate with a temperature in excess of 50 F (10 C).

3.7 — Pumping structural lightweight concrete

3.7.1 *General considerations* — The type of aggregate discussed generally have a surface texture that can vary from angular crushed to rounded coated. In general, they have the following in common:

- Made up of non-connected voids
- Bulk saturated specific gravities of 1.10 to 1.60
- Top size of 3/4 in. (19 mm)

The ability of the lightweight aggregate to absorb relatively large weights of water in 24 hr is the main reason for the difficulty in pumping structural lightweight. For this reason it is of primary importance to presoak or presaturate the lightweight aggregate before pumping. The presaturating can be accomplished by any of the following:

A. ATMOSPHERIC: This can be accomplished by using a soaker hose or sprinkler system. A minimum of 24 hr should be allowed with 72 hr or more preferred. This is dependent on the rate of absorption of the aggregate so the supplier should be consulted. This can be done at the aggregate plant or batch plant.
B. THERMAL: Is accomplished by immersion of hot aggregate in water. Must be carefully controlled and can only be done at the aggregate plant.
C. VACUUM: Is accomplished by introducing dry aggregate into a vessel from which the air can be evacuated. The vessel is then filled with water and returned to atmospheric pressure. This also is recommended for the aggregate plant only. (This method is covered by a patent.)

Presaturation minimizes the ability of the aggregate to absorb water, therefore, minimizing the slump loss during pumping. This additional moisture also increases the loose density of the lightweight aggregate which in turn increases the density of the plastic concrete. This increased weight due to presaturation will eventually be lost to the atmosphere in drying and provides for additional internal curing.

3.7.2 *Proportioning pump mixes* — When considering pumping of lightweight aggregate, it should be taken into account that some adjustments may be necessary to achieve the desired finished product. The architect, engineer, and contractor should be familiar with any adjustments required before the decision is made as to the method of placement. The ready mix producer and aggregate supplier should be consulted so that the best possible pump mixture can be determined. It is at this time the architect and engineer can decide if his project specifications will allow changes, if any, that may be needed to accommodate pumping.

Assuming the project specifications will allow pumping there are general rules that apply. These are based on the use of lightweight coarse aggregate and natural sand fine aggregate.

A. Presaturate lightweight by any of the methods given above.
B. Maintain a 564 lb per cu yd minimum cement content.
C. Use any admixtures that will aid in pumping.
   1. Air entrainment sufficient for 5 to 8 percent air.
   2. Water reducer.
   3. Fly ash or natural pozzolan.
   4. Pumping aid.
D. To facilitate pumping, adjustments in the standard mix proportion usually consists of some slight reduction in the volume of coarse aggregate, with a corresponding increase in the volume of fine aggregate.
E. Cementitious content should be sufficient to accommodate a 4 in. to 6 in. slump.
F. Use a natural sand that is well graded with the fineness modulus preferably between 2.2 and 2.7. Consider the possible addition of a fine sand if this F.M. is not available.
G. Use a properly combined coarse and fine aggregate gradation by volume that will prevent the paste from being squeezed through the voids between aggregate particles. The gradation comparison should be made by volume rather than by weight to account for differences in specific gravity of various particle sizes.

It should be noted that it may sometimes be advisable to plan on various mixture designs as the height of a structure or distance from the pump to the point of discharge changes. Final evaluation of the concrete should be made at discharge end of the pumping system, as suggested by ACI 304.[16]

3.7.3 *Pump and pump system* — After the above items are discussed and implemented the most important function has yet to be completed — pumping of the concrete. Listed below are some of the key items pertinent to the pump and pumping system.

A. Use the largest size line available, preferably a minimum of 5 in.
B. All lines should be clean, the same size, and buttered with grout.
C. Avoid rapid reduction from the pump to line. For example, 10 in. to 4 in. in 4 ft will not work as well as 10 in. to 6 in. in 8 ft, then 6 in. to 4 in. in 4 ft.
D. Reduce the operating pressure by:
   - Slowing down rate of placement.
   - Using as much steel line and as little rubber line as possible.
   - Limiting the number of bends.
   - Making sure the lines are tightly joined and gasketed.

A field trial should be run using the pump and mix design intended for the project. Those present should include representatives of the contractor, ready-mix producer, architect and engineer, pumping service, testing agency and aggregate supplier. In the pump trial, the height and length the concrete is to be moved should be taken into account. Since most locations will not allow the concrete to be pumped as high as it would during the project, the following rules of thumb can be applied for the horizontal run with steel line.

1.0 ft vertical	=	4.0 ft horizontal
1.0 ft rubber hose	=	2.0 ft of steel
1.0 ft 90 degree bend	=	3.0 ft of steel

3.8 — Laboratory and field control

Changes in absorbed moisture or density of lightweight aggregates (which result from variations in initial moisture content, gradation, or specific gravity) and variations in entrained air content[9b] suggest frequent checks of the fresh concrete at the job site to assure consistent quality. Sampling should be in accordance with ASTM C172.[8e] Four simple tests are normally required: (a) standard slump test, ASTM C143,[8f] (b) unit weight of the fresh concrete, ASTM C567,[8g] (c) entrained air content, ASTM C173,[8h] and (d) compressive strength, ASTM C31.[8i]

At the job start, the plastic properties, unit weight, air content, and slump, of the first batch or two should be determined to verify that the concrete conforms to the laboratory mix. Adjustments, if necessary, may then be made immediately. In general when variations in fresh unit weight exceed ±2 percent, an adjustment in batch weights may be required to restore the concrete properties specified. The air content of lightweight concrete, should not vary more than ±1.5 percentage points from a specified value to avoid adverse effects on compressive strength, workability, or durability, (see Section 3.2.1.5).

Chapter 4 — Physical and mechanical properties of structural lightweight aggregate concrete

4.1 — Scope

This chapter presents a summary of the properties of structural lightweight aggregate concrete. The information is based on many laboratory studies as well as a large number of existing structures that have provided satisfactory service over the years.[19-49,52-69]

The customary requirements for structural concrete are that the mix proportions should be based on laboratory tests or on mixes with previous records of performance indicating that the proposed combinations of ingredients will perform as required. The data that are presented may be considered the properties anticipated from properly designed mixtures of sound materials placed and cured in accordance with recognized good practice.

4.2 — Method of presenting data

In the past, properties of lightweight concrete have been compared with those of normal weight concrete, and usually the comparison has been with a single normal weight material. With several million cubic yards of structural lightweight concrete being placed each year, a comparison of properties is usually no longer considered necessary. With the numerous structural lightweight aggregates available, it is as difficult to furnish absolute property values as it is for normal weight concretes made from countless aggregates. For this reason, the data on various properties are presented as the reasonable conservative values to be expected in relationship to some fixed property such as compressive strength, unit weight, or in the case of fire resistance, slab thickness.

References at the end of this chapter consist of laboratory reports as well as papers, suggested guides, specifications and standards. In addition, references that discuss particular structural lightweight concrete structures are included to assist the reader in comprehending the extensive use of structural lightweight aggregate concrete.

4.3 — Compressive strength

Compressive strength levels required by the construction industry for the usual design strengths of cast-in-place, precast or prestressed concrete can be obtained economically with the structural lightweight aggregates in use today.[12,21,26,27,31] Design strengths of 3,000 to 5,000 psi (20.68 to 34.47 MPa) are common. In precast and prestressing plants design strengths of 5,000 psi (34.47 MPa) are usual.

All aggregates have strength ceilings and with lightweight aggregates the strength ceiling generally can be increased at the same cement content and slump by reducing the maximum size of the coarse aggregate. For example with a particular lightweight aggregate the ceiling might be 5,500 psi (37.92 MPa) with a ¾ in. (19 mm) top size of coarse material. By reducing the top size to ½ in. (12.5 mm) or 3/8 in. (9.5 mm) the ceiling might be increased to 6,500 (44.81 MPa) or in excess of 7,000 psi (48.25 MPa).

The compressive strength of lightweight aggregate is usually related to cement content at a given slump rather than water-cement ratio. Water reducing or plasticizing admixtures are frequently used with lightweight concrete mixtures to increase workability and facilitate placing and finishing.

In some cases, compressive strength can be increased with the partial replacement of lightweight fine aggregate with a good quality of natural sand.[34,36] The aggregate producer should be consulted.

4.4 — Cement content

The cement and water contents required for a particular strength and slump are not mechanical properties of concrete. Nevertheless, these factors have significant effects on the hardened concrete properties.

With lightweight concrete, mix proportions are generally expressed in terms of cement content at a particular slump rather than by the water-cement ratio. Increasing the mixing water without increasing the cement content will increase slump and also increase the effective water-cement ratio.

The usual range of compressive strengths may be obtained with reasonable cement contents with the lightweight aggregates being used for structural applications today. Generally air entraining admixtures are found advantageous. The following table which is based on a number of tests of job concretes suggests the range of cement contents for 28-day compressive strengths for concretes with 3 to 4 in. of slump and 5 to 7 percent air contents.

TABLE 4.4 — Approximate relationship between average compressive strength and cement content

Compressive strength psi	(MPa)	Cement content lbs/cu yd (kg/m³) All-lightweight	Sanded lightweight
2500	(17.24)	400-510 (6,386-8,142)	400-510 (6,386-8,142)
3000	(20.68)	440-560 (7,025-8,941)	420-560 (6,705-8,941)
4000	(27.58)	530-660 (8,462-10,537)	490-660 (7,822-10,537)
5000	(34.47)	630-750 (10,058-11,974)	600-750 (9,579-11,974)
6000	(41.37)	740-840 (11,814-13,410)	700-840 (11,175-13,410)

Specified Notes: (1) For compressive strengths of 3000 psi (20.68 MPa) or less, in order to obtain proper qualities for finishing, cement contents may be higher than necessary for the compressive strength. (2) For compressive strengths in excess of 5000 psi (34.47 MPa), the aggregate producer should be consulted for specific recommendations. Type of cement, method of curing, types of admixtures, extent of mix controls, etc., all have a bearing on the cement content-compressive strength relationship. This table is offered merely as a guide, and the aggregate producer should be consulted for more specific recommendations.

4.5 — Unit weight

Weight reduction for concrete of structural quality is the primary advantage of lightweight concrete. Depending upon the source of material, structural grade lightweight concrete can be obtained in a weight range of 90 to 115 lb/ft³ (1440 to 1840 kg/m³).

Producers of structural lightweight aggregate stock the material in various size fractions. Each producer usually is able to furnish at least the standard sizes of coarse, intermediate and fine aggregate. ASTM limits the weight of the coarse fractions . . . the first three . . . to 55 pcf, (880 kg/m³) and the sand or fine fraction to 70 pcf (1120 kg/m³) dry loose basis. Generally the coarse fractions weight from 38 to 53 pcf (608 to 848 kg/m³) with the larger top size being the lighter for a particular source of material. The sand size will generally range from 50 to 68 pcf (800 to 1088 kg/m³).

By combining two or more of these size fractions or by replacing some or all of the fine fraction with a good local normal weight sand weighing from 95 to 110 pcf, (1520 to 1760 kg/m³) a weight range of concrete of 10 to 15 pcf (160 to 240 kg/m³) can be obtained. The aggregate producer is the best source of information for the proper combinations to achieve a specific unit weight for a satisfactory structural lightweight concrete.

With a particular lightweight aggregate, natural sand replacement will increase the unit weight at the same compressive strength by about 5 to 10 pcf (80 to 160 kg/m³). With the same source of material the additional cement required will increase the weight of 5000 psi (34.47 MPa) concrete over 3000 psi (20.68 MPa) concrete approximately 3 to 6 pcf (48 to 96 kg/m³).

4.6 — Modulus of elasticity

The modulus of elasticity of concrete depends on the relative amounts of paste and aggregate and the modulus of each constituent.[58,59] Sand and gravel concrete has a higher E because the moduli of sand and gravel are greater than the moduli of structural lightweight aggregates. Fig. 4.6 gives the range of modulus of elasticity values for structural all-lightweight concrete and for sand-lightweight concrete. Generally the modulus of elasticity for structural lightweight concrete is considered to vary between ½ to ¾ that of sand and gravel concrete of the same strength. Variations in lightweight aggregate gradation usually have little effect on modulus of elasticity if the relative volumes of cement paste and aggregate remain fairly constant.

The formula for $E_c = w_c^{1.5}\,33\sqrt{f_c'}$ ($w_c^{1.5}\,0.043\sqrt{f_c'}$) given in the ACI Code,[80] may be used for values of w between 90 and 155 pcf (1440 and 2480 kg/m³). Further discussion of this formula is given in Section 5.3. Concretes in service may comply with this formula only within ±15 to 20 percent. An accurate evaluation of E_c may be obtained for a particular concrete by laboratory test in accord with the methods of ASTM C469.[8j]

4.7 — Poisson's ratio

Tests[49] to determine Poisson's ratio of lightweight concrete by resonance methods showed that it varied only slightly with age, strength or aggregate used and that the values varied between 0.16 and 0.25 with the average being 0.21. Tests to determine Poisson's ratio by the static method for lightweight and sand-and-gravel concrete gave values that varied between 0.15 and 0.25 and averaged 0.20. Dynamic tests yielded only slightly higher values.

While this property varies slightly with age, test conditions, concrete strength and aggregate used, a value of 0.20 may be usually assumed for practical design purposes. An accurate evaluation may be obtained for a particular concrete by laboratory test according to the methods of ASTM C469.[8j]

4.8 — Creep

Creep[85-89] is the increase in strain of concrete due to a sustained stress. Creep properties of concrete may be either beneficial or detrimental, depending on the structural conditions. Concentrations of stress, either compressive or tensile, may be reduced by stress transfer through creep, or creep may lead to excessive long-time deflection, prestress loss, or loss of camber. The effects of creep along with those of drying shrinkage should be considered and, if necessary, taken into account in structural designs.

4.8.1 *Factors influencing creep* — Creep and drying shrinkage are closely related phenomena that are affected by many factors, such as: type of aggregate, type of cement, gradation of aggregate, water content of the mix, moisture content of aggregate at time of mix, amount of entrained air, age at initial loading, magnitude of applied stress, method of curing, size of specimen or structure, relative humidity of surrounding air, and period of sustained loading.

4.8.2 *Normally cured concrete* — Fig. 4.8.2 shows the range in values of specific creep (creep per psi of sustained stress) for normally cured concrete, as measured in the laboratory (ASTM C512),[84] when under constant loads sustained for a period of one year. These diagrams were prepared with the aid of two common assumptions: (a) superposition of creep effects are valid (i.e., creep is proportional to stress within working stress ranges); and (b) shrinkage strains, as measured on nonloaded specimens, may be directly separated from creep strains. The band of creep properties for all-lightweight aggregate concrete is wide for concrete having a low 28-day compressive strength but it sharply decreases as compressive strength increases. The band for sand-lightweight concrete is narrower than that for the all-lightweight concrete for all 28-day compressive strengths.[37] Fig. 4.8.2 suggests that a very effective method of reducing creep of lightweight concrete is to use higher strength concrete. A strength increase from 3000 to 5000 psi (20.68 to 34.47 MPa) reduces the creep of all-lightweight concrete from 20 to 40 percent.

4.8.3 *Steam cured concrete* — Several investigations have shown that creep may be significantly reduced by low pressure curing and very greatly reduced by high pressure steam curing. Fig. 4.8.3 shows that the reduction for low pressure steamed concrete may be from 25 to 40 percent of the creep of similar concretes subjected only to moist curing. The reduction for high pressure steamed concrete may be from 60 to 80 percent of the creep of similar concretes subjected only to moist curing. High pressure steam cured concrete has the lowest creep values and the lowest prestress loss due to creep and

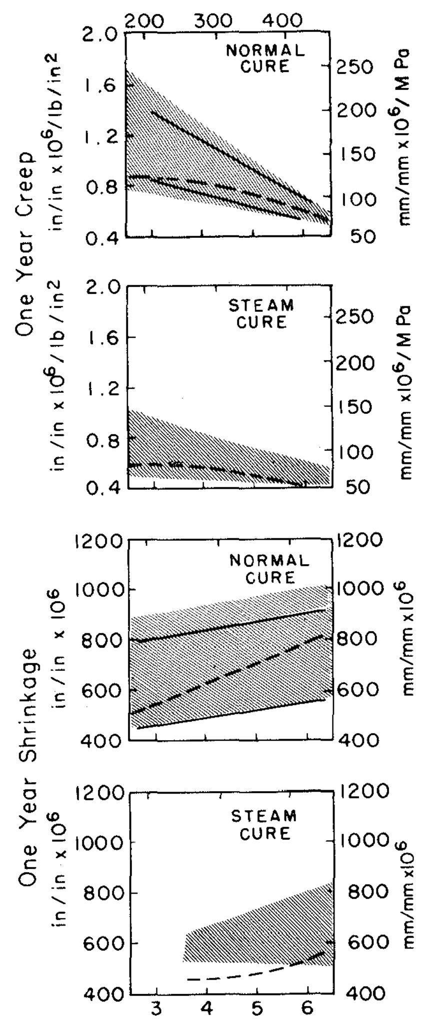


Fig. 4.8.2

Fig. 4.8.3

Fig. 4.9.1

Fig. 4.9.2

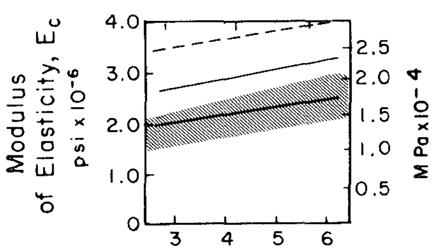
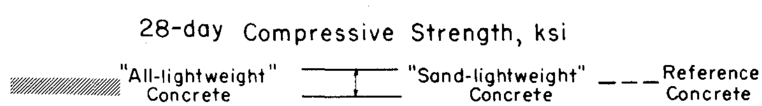


Fig. 4.6



Properties of lightweight concrete

Fig. 4.8.3 — Creep-steam cured concrete

Fig. 4.6 — Modulus of elasticity

Fig. 4.9.1 — Drying shrinkage-normally cured concrete

Fig. 4.8.2 — Creep-normally cured concrete

Fig. 4.9.2 — Drying shrinkage-steam cured concrete

Fig. 4.10.1

Fig. 4.12

Fig. 4.10.2

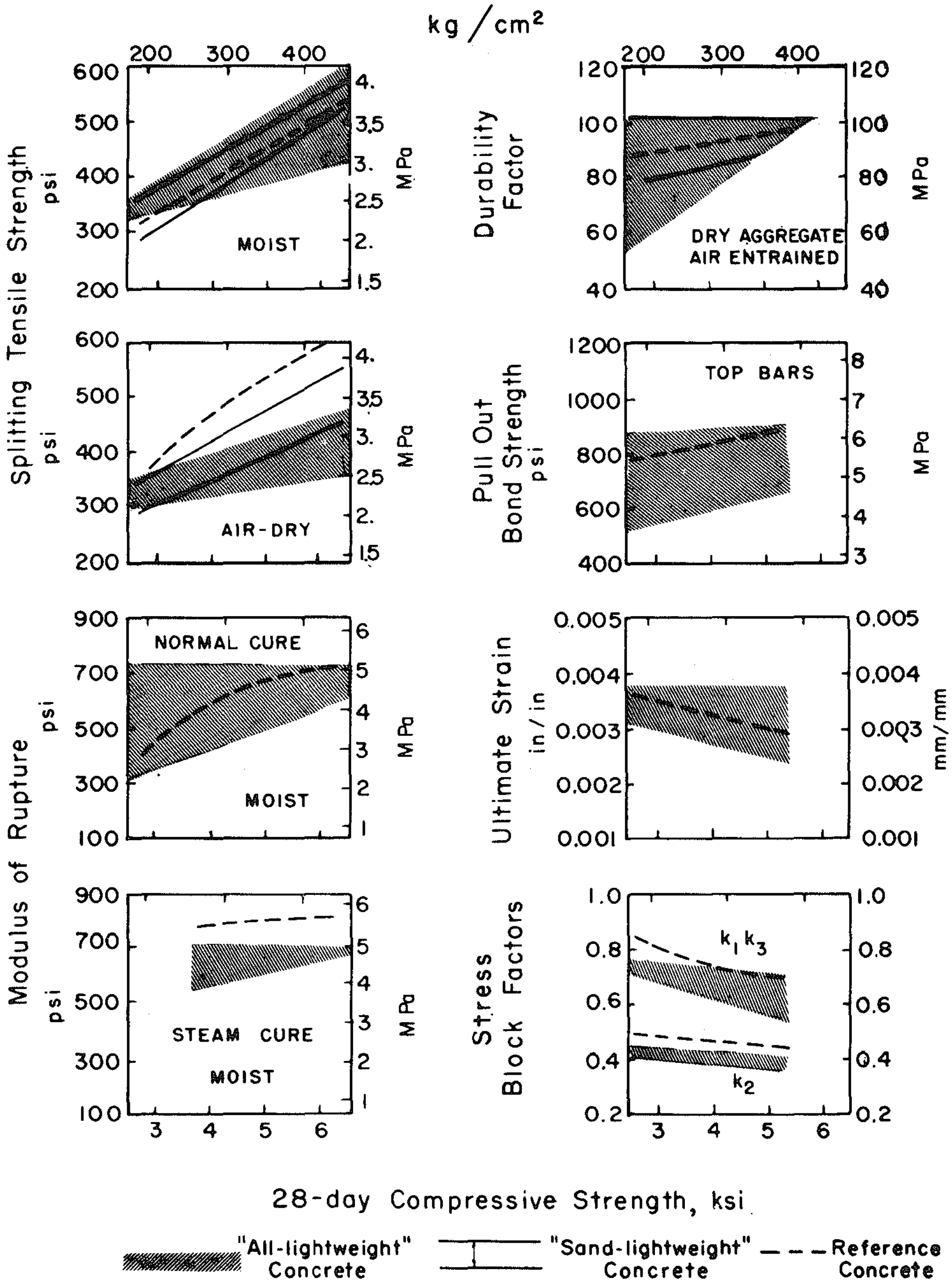
Fig. 4.13

Fig. 4.11(a)

Fig. 4.14.1

Fig. 4.11(b)

Fig. 4.14.2



Properties of lightweight concrete

Fig. 4.10.1 — Splitting tensile strength-moist cured concrete

Fig. 4.10.2 — Splitting tensile strength-air dried concrete

Fig. 4.11(a) — Modulus of rupture-normally cured concrete

Fig. 4.11(b) — Modulus of rupture-steam cured concrete

Fig. 4.12 — Durability factors-freezing and thawing

Fig. 4.13 — Bond strength-pullout tests

Fig. 4.14.1 — Ultimate strain

Fig. 4.14.2 — Stress block factors

shrinkage, while moist cured concrete has the highest values.

4.9 — Drying shrinkage

Drying shrinkage, is an important property that affects extent of cracking, prestress loss, effective tensile strength, and warping. It should be recognized that large-size concrete members, or those in high ambient relative humidities, may undergo substantially less shrinkage than that exhibited by small laboratory specimens stored at 50 percent relative humidity.[81]

4.9.1 *Normally cured concrete* — Fig. 4.9.1 indicates wide ranges of shrinkage values after one year of drying for all-lightweight and sand-lightweight concretes. Noting the position within these ranges of the reference concrete, it appears that low-strength lightweight concrete generally has greater drying shrinkage than that of the reference concrete. At higher strengths, however, some lightweight concretes exhibit lower shrinkage. Partial or full replacement of the lightweight fines by natural sand usually reduces shrinkage for concretes made with most lightweight aggregates.[30,34,37]

4.9.2 *Atmospheric steam cured concrete* — Fig. 4.9.2 demonstrates the reduction of drying shrinkage obtained through steam curing.[23,28,49] This reduction may vary from 10 to 40 percent. The lower portion of this range is not greatly different from that for the reference normal weight concrete.

4.10 — Splitting tensile strength

The splitting tensile strength[22] of concrete cylinders (ASTM C496)[8m] is a convenient relative measure of tensile strength. The test is performed by application of diametrically opposite compressive loads to a concrete cylinder laid on its side in the testing machine. Fracture or "splitting" occurs along the diametral plane. The splitting tensile strength is obtained by use of the following formula:

$$f'_{ct} = \frac{2P}{\pi DL} \qquad \frac{13{,}770\ P}{\pi DL}$$

where

f'_{ct} = splitting tensile strength, psi or Pa
P = total applied load, lb or N
D,L = diameter and length of cylinder, respectively, in. or m

4.10.1 *Moist cured concrete* — Fig. 4.10.1 indicates a narrow range of this property for continuously moist cured lightweight concretes. The splitting tensile strength of the normal weight reference concrete is nearly intermediate within these ranges. Replacement of lightweight fine aggregate by sand has little or no effect on this property. It thus may be concluded[24,35] that the tensile strength for continuously moist cured lightweight concretes is correlated mainly with the compressive strength and may be considered equal to that of equal compressive strength normal weight concrete.

4.10.2 *Air dried concrete* — The tensile strength of lightweight concretes which undergo drying is more relevant in respect to behavior of concrete in structures. During drying of the concrete, moisture loss progresses at a slow rate into the interior of concrete members, resulting in the probable development of tensile stresses at the exterior faces and balancing compressive stresses in the still moist interior zones. Thus the tensile resistance to external loading of drying lightweight concrete will be reduced from that indicated by continuously moist cured concrete.[24,35,75] Fig. 4.10.2 indicates this reduced strength for concretes that have been moist cured 7 days followed by 21 days storage at 50 percent relative humidity (ASTM C330).[8a] The splitting tensile strength of all-lightweight concrete varies from approximately 70 to 100 percent that of the normal weight reference concrete when comparisons are made at equal compressive strength.

Replacement of the lightweight fines by sand generally increases the splitting tensile strength of lightweight concrete subjected to drying.[25,35,67] In some cases[35] this increase is nonlinear with respect to the sand content so that with some aggregates partial sand replacement is as beneficial as complete replacement.

Splitting tensile strength is of particular value for estimating the diagonal tension resistance of lightweight concrete in structures. Tests[24] have shown that the diagonal tension strengths of beams and slabs correlate closely with this property of the concrete.

4.11 — Modulus of rupture

The modulus of rupture (ASTM C78)[8n] is also a measure of the tensile strength of concrete. Fig. 4.11 (a) and 4.11 (b) indicate ranges for normally cured and steam cured concretes, respectively, when tested in the moist condition. Similar to the indications for splitting tensile strength, the modulus of rupture of moist cured lightweight concrete[21,24,75] appears little different from that of normal weight concrete. A number of studies[24,75] have indicated that modulus of rupture tests of concretes undergoing drying are extremely sensitive to the transient moisture content, and under these conditions may not furnish data that is satisfactorily reproducible.

4.12 — Durability

Freezing and thawing durability and salt-scaling resistance of lightweight concrete are important factors, particularly in horizontally exposed concrete construction such as access ramps, exposed parking floors, or bridge decks. Generally, deterioration is not likely to occur in vertically exposed members such as exterior walls or exposed columns, except in areas where these structures are continually exposed to water. As in normal weight concretes, it has been demonstrated that air entrainment provides a high degree of protection to lightweight concretes exposed to freezing and thawing and salt environments.[39,74]

Fig. 4.12 indicates the range of durability factors (similar to that defined in ASTM C-666),[8p] for all-lightweight concretes and for sand-lightweight concretes. The durability factor is the percent of the dynamic modulus of elasticity retained after 300 cycles of freezing and thawing. Some of the concretes shown in the Fig. 4.12 had relatively poor freeze-thaw resistance in the lower strength ranges. Generally these concretes have high water-cement ratios, thus the quality of the cement paste is poor. The same concretes had a much improved rating at higher strengths (lower water-cement ratio). Many lightweight concretes, as shown, can perform equivalent to or better than normal weight concretes. Limited salt-scaling tests have indicated similar satisfactory performance. Natural sand provides for additional resistance at all strength levels. However, the difference in the resistance of air-entrained all-lightweight and sand-lightweight concretes having compressive strengths higher than 5000 psi (34.47 MPa) is small.[36]

The use of water-saturated aggregates (approaching the 24 hr water absorption) at the time of mixing generally reduces freezing and thawing resistance of lightweight concrete. Under some conditions air-entrainment will improve the durability of concrete made with these saturated aggregates. However, experience has shown that as such concretes are allowed to dry, durability improves considerably. If freezing and thawing resistance is required in lightweight concretes, and if it cannot undergo drying prior to freezing exposure, the moisture content of the aggregate should be minimized.

4.13 — Bond strength (pull-out tests)

Field performance has indicated satisfactory behavior of lightweight concrete with respect to bond. The bond strength of lightweight concrete to steel reinforcement, as measured by pull-out strength of reinforcing bars [ASTM C234[8q] top bars, for 0.01 in. (0.25 mm) slip] has usually been measured for all-lightweight concretes.[21,60] Fig. 4.13 indicates the range in results for a somewhat limited number of tests. These tests simulated the conditions of top reinforcing bars in beams and slabs. The bond of bottom bars is generally higher in concrete. Further, this test is made only on a single bar, whereas in actual structures the reinforcement consists of an assemblage. If slip should occur with one bar in this assemblage, stress can be transferred to other bars. Considering the tensile strength of lightweight concrete, precaution should be exercised to investigate the length of reinforcement anchorage in those areas where bond is critical. Flexural bond of lightweight concrete is no different than that of normal weight concrete.

4.14 — Ultimate strength factors

4.14.1 *Ultimate strain* — Fig. 4.14.1 indicates a range of values for ultimate compressive strain for all-lightweight concretes. These data were measured on unreinforced specimens eccentrically loaded to simulate the behavior of the compression side of a reinforced beam in flexure.[19,20] The data indicated for the normal weight reference concrete were obtained in the same manner. This diagram indicates that the ultimate compressive strain of most lightweight concretes (and of the reference normal weight concrete) may be somewhat greater than the value of 0.003, assumed for design purposes. Further studies of the ultimate strain of structural lightweight concrete are under way.

4.14.2 *Stress block factors* — Fig. 4.14.2 presents coefficients relating to an assumed curvilinear stress block at ultimate flexural load.[19,20] These values were obtained simultaneously with the ultimate strains discussed in Section 4.14.1. The factor k_1k_3 represents the ratio of the average stress in the stress block to the cylinder strength of the concrete, and k_2 is the ratio of the depth to the stress block centroid and the depth to the neutral axis. For general design purposes individual values of these coefficients may have little significance.

4.15 — Water absorption of concrete

Generally, lightweight concretes have considerably higher water absorption values than do normal weight concretes. High absorption, however, does not necessarily indicate that concretes will have poor durability or high permeability. Various investigations have failed to reveal any consistent relationship between water absorption of concrete and its durability.[30] The durability of lightweight concrete, as with normal weight concrete, is primarily a function of the cement paste quality, amount of air entrained in the cement paste, and the quality of the aggregate itself. Permeability depends primarily on the quality of the cement paste.

4.16 — Alkali-aggregate reaction

Laboratory studies[5,21] concerning potential alkali-aggregate reactivity of structural lightweight aggregates have indicated little or no detrimental reaction between the alkalis in the concrete and silica in the aggregates. At least half of a typical shale, for example, is silica (a) but occurs as well crystallized silicates and free quartz rather than the nearly amorphous forms of silica such as (b) opal and chalcedony known to be reactive.

4.17 — Thermal expansion

Only a few determinations[5,42,50] have been made of linear thermal expansion coefficients for structural lightweight concrete. Approximate values are 4 to 6

$\times 10^{-6}$ in./in./F (7 to 11 $\times 10^{-6}$ mm/mm/C) depending on the amount of natural sand used.

Ranges for normal weight concretes are 5 to 7 $\times 10^{-6}$ in./in./F (9 to 13 $\times 10^{-6}$ mm/mm/C) for those made with siliceous aggregates and 3.5 to 5 $\times 10^{-6}$ in./in./F (6 to 9 $\times 10^{-6}$ mm/mm/C) for those made with limestone aggregates[70] the values in each case depending upon the mineralogy of specific aggregates.

4.18 — Heat flow properties

4.18.1 *Thermal conductivity* — The value of thermal conductivity, k, is a specific property of a material (rather than of a construction) and is a measure of the rate at which heat (energy) passes perpendicularly through a unit area of homogeneous material of unit thickness for a temperature gradient of one degree:
U.S. units, k = Btu/hr ft² (deg F/in.)
(S.I. units, k = $W/m{\cdot}K$)

Thermal resistivity is the resistance per unit of thickness and is equal to $1/k$.

Thermal conductivity has been determined for concretes ranging in oven-dry density from less than 20 to over 200 pcf (320 to 3200 kg/m³).* Conductivity values are obtained according to ASTM C177[85] guarded hot plate on specimens in an oven-dry condition.

When k values for concretes having a wide range of densities are plotted against oven-dry density, best-fitting curves show a general dependence of k on density, as shown in Fig. 4.18.1, originally published in 1956.[71] Also shown is the fact that different investigators have provided different relationships. These differences are accounted for by differences in materials, particularly in aggregate mineralogical type and microstructure, and in gradation. Differences in cement content, and matrix density and pore structure also occur. Some differences in test methods and specimen sizes also existed.

Valore[72] plotted over 400 published test results of density against the logarithm of conductivity and suggested the equation:
$k = 0.5\ e^{0.02w}$ ($k = 0.072\ e^{0.00125w}$)
Existing data in the ASHRAE Handbook of Fundamentals 1977[73] compares very closely with the suggested formula. An accurate k value for a given concrete, based on testing by the method of ASTM C177 is preferable to an estimated value, but for purposes of estimation, the formula provides a good base for estimating k for concrete in the oven-dry condition and, in addition, may easily be revised for air-dry conditions.

4.18.2 *Effect of moisture on thermal conductivity of concrete* — It is generally acknowledged that increasing the free moisture content of hardened con-

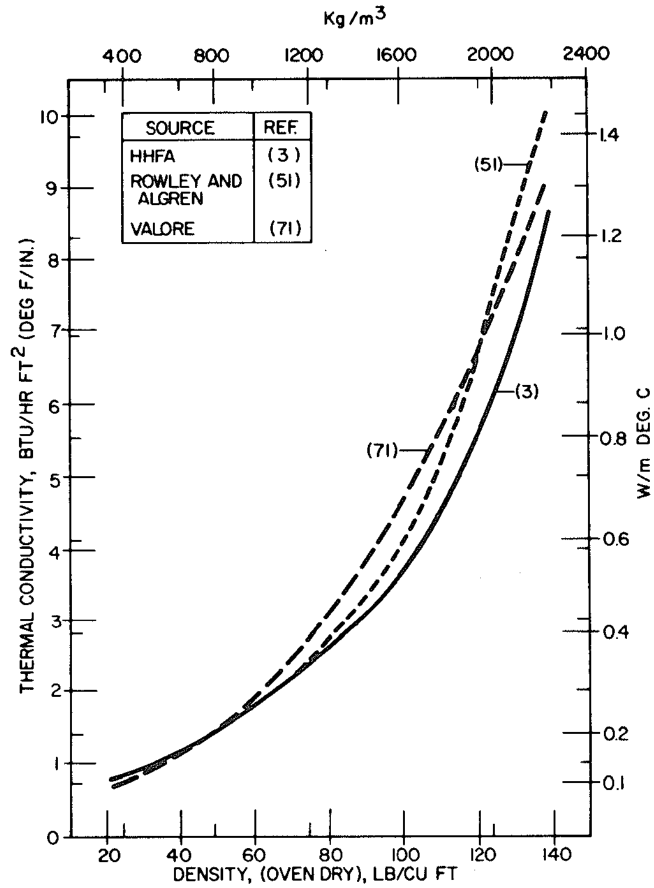


Fig. 4.18.1 — Relation of average thermal conductivity, k, values of concrete in oven-dry condition to density

crete causes an increase in thermal conductivity. In Reference 72, a rule-of-thumb was stated that k increases by 6 percent for each one percent increment in free or evaporable moisture, by weight in relation to oven-dry density. k (corrected) =

$$k\ (\text{oven-dry}) \times \left(1 + 6\ \frac{(w_m - w_o)}{w_o}\right)$$

where w_m and w_o are densities in moist and oven-dry conditions, respectively.

Data on the effect of moisture on k of lightweight aggregate concretes are mostly of European origin and have been summarized by Valore.[72]

4.18.3 *Equilibrium moisture content of concrete* — Concrete in a wall is not in an oven-dry condition; it is in an air-dry condition. Since k values shown are for oven-dry concrete, it is necessary to know the moisture content for concrete in equilibrium with its normal environment in service and then apply a moisture correction factor for estimating k under anticipated service conditions. While relative humidity within masonry units in a wall will vary with type of occupancy, geographical location, exposure, and with the seasons, it may be assumed to be a constant relative humidity of 50 percent. It is further assumed

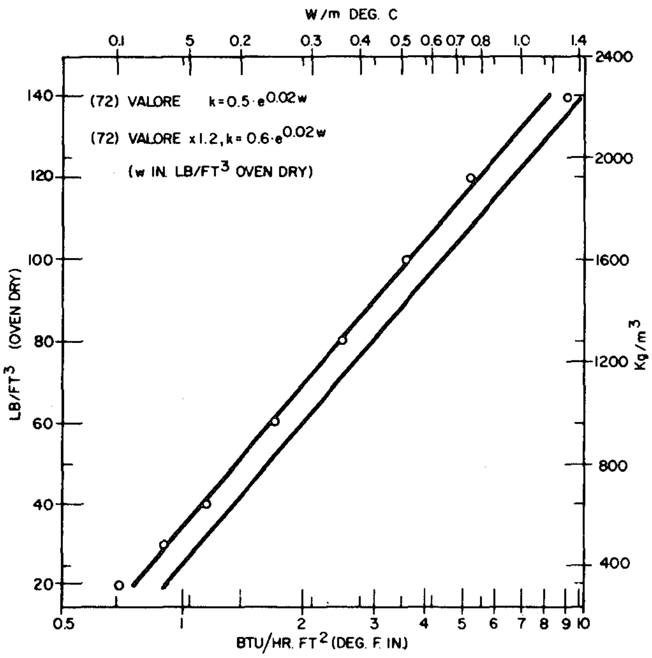


Fig. 4.18.2 — Relation of average k values of concrete to dry density

that exterior surfaces of single-wythe walls are "protected" by paint (of a "breathing" type), stucco or surface bonding fibered cement plaster. For single-wythe walls, such protection is necessary to prevent rain penetration. For cavity walls, the average moisture content of both wythes, even with the exterior wythe unpainted, will be approximately equal to that of the protected single-wythe wall.

Data from various sources for structural sand-gravel and expanded shale concretes, and for low density insulation concretes have been summarized in References 71, 72, and 73. Average long-term moisture contents for structural concretes are in good agreement with data for concrete masonry units.

It is recognized that, under certain conditions, condensation within a wall can cause high moisture contents, and that temperature gradients within the wall cause moisture to migrate to the cold side. Nevertheless, the assumed average values appear to form a reasonable basis for estimating average effects of moisture on k.

4.18.4 *Recommended moisture factor correction for thermal conductivity* — Moisture factors of 6, and 9 percent increase in k per 1 percent of moisture, by weight, are recommended for lightweight aggregate concretes (of all types) and normal weight concrete, respectively. These factors are for use where exposure conditions or other factors produce moisture contents known to depart appreciably from recommended standard moisture contents of 2 percent, for normal concrete, and 4 percent (by volume) for lightweight concrete.

As a practical matter, a simple constant factor can be used for masonry unit and structural concretes, under conditions of normal protected exposure. The recommended factor to be multiplied by k values of oven-dry concrete is 1.2; i.e., k values corrected for equilibrium moisture in normal protected exposure are to be increased by 20 percent over standard values for oven-dry concrete. This correction factor is recommended for application to the Valore equation of Fig. 4.18.2, which now becomes: $k = 0.6e^{0.02w}$, in Btu/hr ft² (deg F/in.) ($k = 0.0865e^{0.00125w}$, in $W/m \cdot K$), in that figure, where $e = 2.71828$ and w is the density of concrete, oven-dry in pcf and kg/m³, respectively.

4.18.5 *Cement paste as insulating material* — The oven-dry density of mature portland cement paste ranges from 100 pcf (1600 kg/m³) for a water-cement weight ratio of 0.4, to 67 pcf (1075 kg/m³) for a w/c ratio of 0.8. This range for w/c ratio encompasses structural concretes. Campbell-Allen and Thorne [78] and Lentz and Monfore[42] have studied the influence of cement paste on k of concrete. The former study provided theoretical k values for oven-dry and moist pastes and the latter reported measured k values of water-soaked pastes. Other data on moist-cured neat cement cellular concretes (aerated cement pastes) permit us to develop k-density relationships for oven-dry, air-dry, and moist pastes.[71] The latter work shows that neat cement cellular concrete and autoclaved cellular concrete follow a common k-density curve.

4.18.6 *Thermal transmittance* — U-value is thermal transmittance; it is a measure of the rate of heat flow through a building *construction*, under certain specified conditions. It is expressed in the following units: U = Btu/hr ft² deg F ($W/m^2 \cdot K$).

The U-value of a wall or roof consisting of homogeneous slabs of material of uniform thickness is calculated as the reciprocal of the sum of the thermal resistance of individual components of the construction:

$$U = \frac{1}{R_1 + R_2 + R_3 + \ldots\ldots\ldots\ldots R_n}$$

where R_1, R_2 etc. are resistances of the individual components and also include standard constant R values for air spaces, and interior and exterior surface resistances. R is expressed in the following units:

R = deg F/(Btu/hr ft²), ($R = m^2 \cdot K/W$).

Thermal resistances of individual solid layers of a wall are obtained by dividing the thickness of each layer by the thermal conductivity, k, for the particular material of which the layer consists.

4.19 — Fire endurance

Structural lightweight concretes are more fire resistant than normal weight concretes because of their lower thermal conductivity, lower coefficient of thermal expansion and the inherent fire stability of an aggregate already burned to over 2000 F (1100 C).*

4.19.1 *Heat transmission* — Recent research on fire endurance comparing lightweight aggregate concrete with normal weight concrete all with f_c' = 4000 psi (27.58 MPa)[40] yielded the data shown in Fig. 4.19.1. In these tests the lightweight aggregate concrete would be classified as sand-lightweight.

4.19.2 *Cover requirements* — The thickness of concrete between reinforcing steel (or structural steel) and the nearest fire-exposed surface is called "cover." For simply supported slabs, beams and columns, fire endurance is dependent largely on cover. The cover requirements for lightweight concrete are slightly lower than those for normal weight concrete.

4.19.3 *Strength retention* — Abrams[41] reported that carbonate aggregate concrete and lightweight concrete tested hot without prior loading retained about 75 percent of their original strengths (strengths prior to heating) at 1200 F (649 C). The sanded lightweight concrete had strength characteristics at high temperatures similar to carbonate concrete.

4.20 — Abrasion resistance

Tests indicating abrasion resistance have been conducted on a limited number of slabs using several commercially available lightweight aggregates. The method of test[83] and the degree of abrasion simulated the wear encountered in public, commercial, and industrial environments. Among other results, these tests confirmed that abrasion resistance of structural lightweight concrete varies with compressive strength in a manner similar to normal weight concrete.

Most lightweight aggregates, acceptable for structural concrete were at one time molten, and on cooling resulted in a vesicular particle having an adequate gross strength. The composition of the solidified material is such that it ranks high on Moh's scale of hardness, often comparable or superior to that of glass, and the equal of quartz, feldspar, or volcanic minerals. However, because of its vesicular structure, the net resistance to load and/or impact may be low compared to a solid particle of similar composition. Therefore, the abrasion resistance of "all-lightweight" concrete may not be suitable for steel-wheeled or exceptionally heavy industrial traffic in commercial establishments. As the severity of wear becomes less, i.e., in light warehousing, markets, public buildings, schools, churches, residences, etc., the abrasion resistance should be as satisfactory as that of normal weight concrete.

The abrasion resistance of structural lightweight concrete can be improved in several ways. First, the

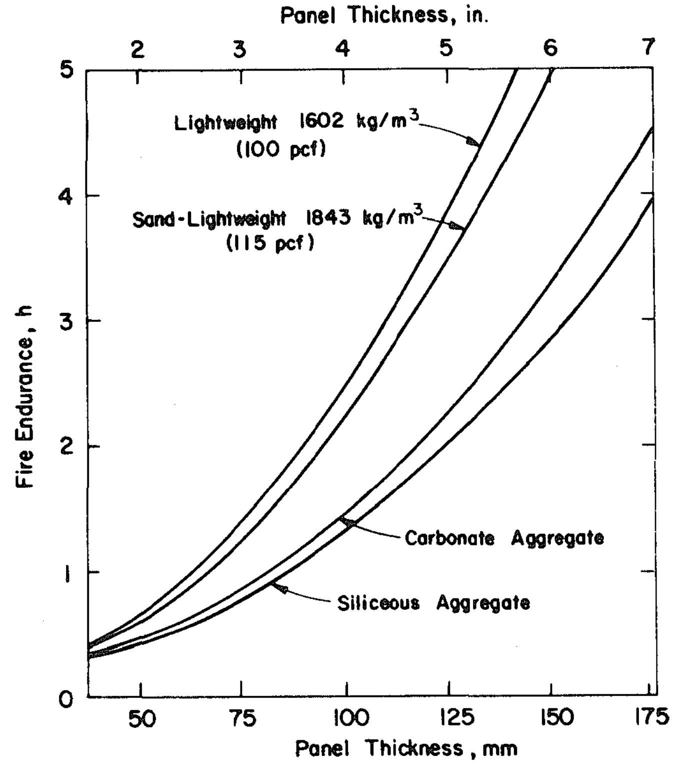


Fig. 4.19.1 — Fire endurance (heat transmission) of concrete slabs as a function of thickness for naturally dried specimens[40]

relatively soft lightweight coarse aggregate can be combined with a hard fine aggregate to resist abrasive wear in a manner similar to combinations of hard fine aggregates and soft natural coarse aggregates such as limestone.

Another approach is to apply a natural sand-portland cement dry shake to the surface of the concrete. Thus, the benefit of low weight of an all-lightweight aggregate concrete is combined with the abrasion resistance of a hard fine aggregate. Further, iron-aggregate shakes have similarly been employed in heavy industrial applications of structural lightweight concrete.

Chapter 5 — Design of structural lightweight aggregate concrete

5.1 — Scope

The availability of quality lightweight aggregates, capable of providing satisfactory structural concrete has led to economical design of lightweight buildings, bridges, and other structures since World War II. During much of this period, designs were based on the fundamental properties of concrete, properly evaluated by the structural engineers, but without the guidance and control of building codes or recommended practices specifically pertaining to structural lightweight concrete. With the adoption of the 1963 ACI Building Code (ACI 318-63) lightweight aggregate concrete received full recognition as an acceptable structural medium. Some general guidelines for

*References 40, 41, 43, 44, 45, 47, 48

the structural engineer and for the construction industry, in general, were presented.

This chapter of the Guide is intended to interpret the present ACI Code (318-77)[80] requirements for structural lightweight concrete. At the same time it condenses many practical design aspects pertaining to lightweight concrete and provides the structural engineer with additional information on which to base engineering judgment.

It is assumed that a structural engineer will obtain information on the properties of concrete made with specific lightweight aggregate (or aggregates) available for a given project. It is also assumed that these aggregates will fall within the frame of reference presented in this Guide, and that the specifications will be prepared so that only suitable structural lightweight aggregates will be used.

5.2 — General considerations

Lightweight aggregate concrete has been shown by test and performance (see Chapter 4) to behave structurally in much the same manner as normal weight concrete, but at the same time to provide some improved concrete properties, notably reduced weight and better insulation. For certain properties of concrete, the differences in performance are those of degree. Now there is often more data available on the engineering and design properties of structural lightweight concrete than on normal weight concrete. The new editions of the ACI Code attempt to modify structural designs in lightweight concrete to achieve approximately the same load factors as for normal weight concrete design. Generally those properties that include tensile strength (see Section 4.10) and modulus of elasticity (see Section 4.6) are sufficiently different from those of normal weight concrete to require design modification.

5.3 — Modulus of elasticity

It has been shown that the modulus of elasticity of concrete is a function of unit weight and compressive strength. The formula, $E_c = w_c^{1.5}\ 33\sqrt{f_c'}$ ($E_c = w_c^{1.5}\ 0.043\sqrt{f_c'}$) presented in the 318-77[80] ACI Code, defines this relationship. Any design calculations for lightweight concrete should be based on the corresponding modulus of, for example, deflection calculations in flexural members, buckling effects in long columns or slender beams, and elastic shortening in arches should be based on the E_c value of lightweight concrete. Variations of the ACI formula for E_c at the high strength used in prestressed concrete are covered in Section 5.12. Depending on how criticaly the values for E_c will affect the nature of the design, the engineer should decide whether the values determined by formula are sufficiently accurate, or whether he should call for determination of E_c values from tests on the specified concrete.

A lower E_c value for lightweight concrete means essentially that it is less stiff, since stiffness is defined as the product of modulus of elasticity and moment of inertia *(EI)*. Reduced stiffness can be beneficial at times, and the use of lightweight concrete should be considered in these cases instead of normal weight concrete. In cases requiring improved impact or dynamic response, where differential foundation settlement may occur, and in certain types or configurations of shell roofs, the property of reduced stiffness may be desirable.

5.4 — Tensile strength

Tensile strength of lightweight concrete, for equal compressive strength, is comparable to that of normal weight concrete when continuously moistcured specimens are tested. Although the mechanisms of drying and the effects of moisture gradients are not fully understood, it is generally recognized that air drying reduces the tensile strength of lightweight concrete (see Section 4.10). For this reason test values for the diagonal tension resistance of lightweight concrete are generally lower, and hence shear design formulas in the ACI Code are modified accordingly. There are a few other instances where tensile strength is important, for example, in allowable cracking stress for prestressed members and determining when deflection calculations should be based on a cracked section instead of a homogeneous section.

5.5 — Development length

Basic development length factors of the ACI Code 318-77[80] reflect the lower tensile splitting strength of structural lightweight concrete. Provisions for modification of development length for all-lightweight and sanded lightweight concretes are similar to the tensile strength sections of the Code. For a full explanation of use of modification factors see Section 5.8 of this Guide.

5.6 — Creep and shrinkage

Values for creep and shrinkage show sufficient range for concretes made with both normal weight and lightweight aggregates so that average, minimum, or maximum values can be used only with qualifying phrases.

Fig. 5.6.1 and 5.6.2 are a summary from NBS Monograph # 74, after a five-year study of high strength concretes highly stressed at early ages. It should be noted that the overlapping of data indicates the "regular" normal weight aggregates do not have the standard properties that lightweights are often compared against. Designs that are based on creep properties or that recognize shrinkage performance fall into this category. Therefore, when these properties are included in design considerations, generalities given in the ACI Code and in other guides to design are subject to engineering judgment, and specific data or performance of job materials are the preferred basis for design. Furthermore, it should be recognized that the effects of creep and shrinkage are moderated by internal stress redistribution. Creep and shrinkage of concrete account for the greater portion of prestressing losses in post-tensioned concrete, but an engineer may be in error by 50 or 100 percent by using an arbitrary stress loss in any given location, whether his aggregate is normal or lightweight. Since creep and shrinkage loss is significant in prestressing design, calculations should be based on test data for specific materials being used. In other design considerations, such as sustained load deflection, the accuracy of assumed creep and shrinkage values is overshadowed by other variables so that they become less significant and the general ACI Code requirements may be adequate.

Investigations into the difference in behavior of structural lightweight and normal weight concrete in columns caused by the effect of creep and shrinkage are covered in detail in Section 5.11.

5.7 — Deflection

5.7.1 *Initial deflection* — Section 9.5 of the Code specifically includes the modifications of formulas and minimum thickness requirements that reflect the lower modulus of elasticity, lower tensile strength and lower modulus of rupture or lightweight aggregate concretes.

The table of Section 9.5 of ACI 318-77[80] listing minimum thickness of beams or one-way slabs unless deflections are computed, requires a minimum increase of 9 percent in thickness for lightweight members over normal weight. Thus, using the values in this table, lightweight structural members with increased thickness are not expected to deflect more than normal weight members under the same superimposed load.

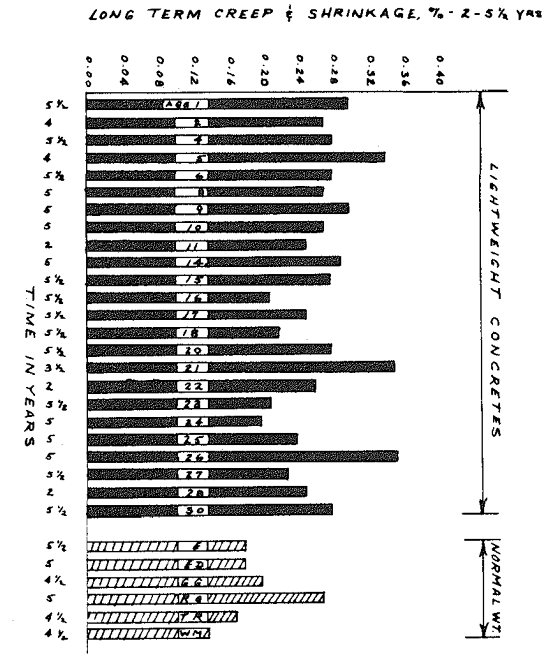


Fig. 5.6.1 — Long term creep and shrinkage

5.7.2 *Long-term deflection* — Analytical studies of long-term deflections can be made, taking into account the effects which occur from creep and shrinkage. Final deflection can then be compared with the initial deflection due to elastic strains only. Com-

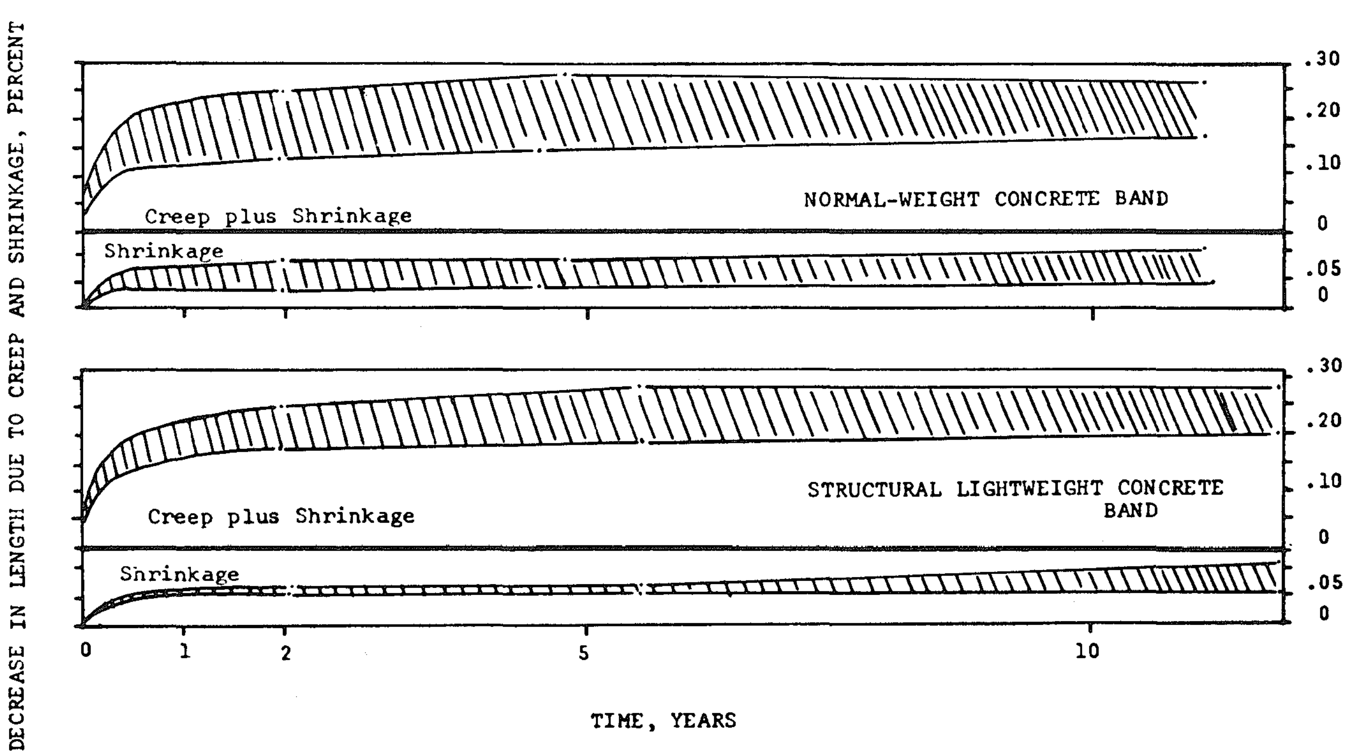


Fig. 5.6.2 — Sample data showing results to date for ESCSI creep program

parative studies of lightweight and normal weight concrete show that the ratio of creep strain is the same for both lightweight and normal weight concrete. Comparative shrinkage values for concrete vary appreciably with variations in component materials, and may even be less for concrete made with a high quality lightweight aggregate than for that made with marginal normal weight aggregate (see Section 4.9). In typical cases, however, the shrinkage of lightweight concrete may be somewhat greater than comparable normal weight concrete. The effect of shrinkage on deflection arises from the restraint of shrinkage due to steel reinforcement. Tests have shown that for usual amounts of reinforcements, the effect of shrinkage on deflection is quite small regardless of type of concrete. Thus, the difference between the shrinkage deflection of lightweight and normal weight members of comparable design must be quite small. Such an analysis of deflection due to elastic strain, creep and shrinkage, leads to the same factor given in Section 9.5.2.5 of ACI 318-77[80] and it is recommended that this factor for obtaining long-term deflections be used for both types of concrete. More refined approaches to estimating deflections are, in general, not warranted.

5.8 — Shear and diagonal tension

Lightweight concrete members, subject to shear and diagonal tension, behave in fundamentally the same manner as normal weight concrete members. In both cases, the shear and diagonal tension capacity of the concrete member is determined primarily on the tensile capacity of an unreinforced web. Since most concrete in construction is subjected to air drying, lightweight concrete will generally have lower tensile strength than normal weight concrete of equal compressive strength (see Section 4.10). ACI 318-77[80] provides two alternate approaches by which the permissible shear capacity in a lightweight concrete member may be determined. The permissible shear capacity may be determined by utilizing the splitting tensile strength f_{ct} for the specific aggregate to be used or by using a fixed percentage of a similar strength normal weight concrete.

Using the first approach to calculate the permissible shear, the value of $f_{ct}/6.7$ is substituted for $\sqrt{f_c'}$ in the provisions of Chapter 11 of the Code. A few lightweight aggregates develop high tensile strength[24, 35] so that the shear performance of concrete members using these aggregates is comparable to similar members of normal weight concrete. The possible shear capacity for lightweight concrete members, however, should never exceed that of the normal weight concrete of the same strength.

Most structural lightweight aggregate producers have sufficient data available to realistically estimate the range of f_{ct} values which can be achieved using all lightweight coarse and natural fine aggregates. A realistic value of f_{ct} for design purposes should be established for each desired compressive strength and composition of concrete. The f_{ct} values on which the structural design is based should be incorporated in the concrete specifications for the job. Splitting cylinder strength tests should also be required by the specification to insure that the design concretes, using the selected aggregate, satisfy the f_{ct} requirement. These tests should be performed in accordance with "Specifications For Lightweight Aggregates For Structural Concrete" (ASTM C330). Once the f_{ct} has been confirmed for the specific concretes to be used on the job, it is not necessary to run additional splitting cylinder tests during construction for quality control purposes. The concrete's compressive strength is an indication of the concrete's splitting tensile strength, and therefore field tests of f_{ct} need not be conducted.

A second, generally conservative, approach in calculating the permissible shear may be used when the engineer is unable or is hesitant to specify f_{ct} values. Reduction factors are available which may be used to determine the shear of lightweight or natural sand lightweight concrete as a fixed percentage of normal weight concrete shear. Research[35] on the splitting tensile strength of lightweight concrete shows some improvement in tensile strength when natural sand is used in place of the lightweight fine aggregate. Two reduction factors have, therefore, been established: 75 percent of normal weight values for all lightweight aggregates; and 85 percent of normal weight values for combinations of natural sand fine aggregates and lightweight coarse aggregates.

Since reduction in dead weight leads to a substantial reduction in total load on lightweight concrete members, shear capacity, reduced to as much as 75 percent that of normal weight concrete, does not necessarily lead to an increase in web reinforcement, or for that matter, a decrease in relative structural efficiency.

5.9 — Strength design

The strength design requirements in the ACI Code[80] for flexural computations and for combined axial compression and bending apply to structural lightweight concrete. Where the code requires a differentiation due to the reduced modulus the equations are suitably modified.

For example, the code assumes the maximum compressive strain in the extreme fiber to be 0.003. Tests[19,20] have shown this to be a reasonably conservative assumption for both normal weight and lightweight concrete. In a similar manner, certain of the basic coefficients, can be shown to apply to both lightweight and normal weight concrete.[20]

The basic philosophy in the design for flexural capacity is that failure will occur by yielding of the steel rather than by crushing of the concrete. The formulas have been prescribed to insure this type of performance, and hence the properties of the concrete, once adequate strength is maintained, are not of major importance to ultimate safety of structures. Tests of lightweight concrete members to failure have verified the ultimate strength design of the members.[44,54]

5.10 — Working stress design

While the Code has essentially de-emphasized what was originally titled working stress design, it still includes this approach as an alternate design method. The difference in concretes caused by the differences in modulus are suitably accounted for.

5.11 — Columns

The design of columns using structural lightweight concrete is essentially the same as for normal weight concrete. The reduced modulus should be used in the code sections in which slenderness effects are considered.

Extensive tests[85,86] comparing the time-dependent behavior of structural lightweight and normal weight columns developed the following facts.

1. Instantaneous shortening caused by initial loading can be accurately predicted by elastic theory. Such shortening of a lightweight concrete column will be greater than that of a comparable normal weight column due to the lower modulus of elasticity of lightweight concrete.
2. Time-dependent shortenings of lightweight and normal weight concretes may differ when small *un*reinforced specimens are compared. However, these differences are minimized when large reinforced concrete columns are tested; both increasing size and longitudinal reinforcements reduce time-dependent shortenings. Measured time-dependent shortenings were compared with those predicted by theory and satisfactory correlations were found.
3. Measured ultimate strengths were compared with theory and good correlations were found. Both concrete type and previous loading had no effect on this correlation.
4. The lightweight concrete columns generally had slightly greater ultimate strain capacity when they were unreinforced. When reinforced, the strain capacities were closely similar.

5.12 — Prestressed lightweight concrete applications

5.12.1 *Applications* — In recent years prestressed lightweight aggregate concrete has been widely used in both North America and Europe. The new material has been found particularly useful in certain building applications and, to some extent, in nearly every application for which prestressed normal weight concrete has been employed. The most beneficial applications are those in which the unique properties of prestressed lightweight aggregate concrete are fully utilized. It is selected not merely as a lightweight substitute for prestressed normal weight concrete but as a new material in its own right.

Prestressed lightweight concrete has been used extensively in roofs, walls, and floors of buildings. Particularly in flat plate construction, prestressed lightweight aggregate concrete has found extensive use. For these uses, the reduced dead weight with its lower structural, seismic and foundation loads, the better thermal insulation and better fire resistance have usually been the determining factors in the selection of prestressed lightweight concrete.[82]

Several newer applications of the material appear promising. Many of these are based on its energy-absorption properties and reduced modulus of elasticity, others on its thermal properties, and still others on its greatly reduced submerged weight.

Prestressed lightweight concrete has been used in composite action with normal weight concrete. Many combinations have been tried and have proved successful structurally. These combinations are:

- Prestressed lightweight aggregate concrete joists and beams with deck slab of normal weight concrete cast-in-place.
- Prestressed lightweight aggregate concrete joists and beams with deck slab of lightweight aggregate concrete cast-in-place.
- Prestressed normal weight concrete beams with cast-in-place lightweight aggregate concrete.
- In general, combinations 1 and 2 are most efficient because of the relative moduli of elasticity. However, combination 3 has proved suitable in many cases including bridge structures.

5.12.2 *Properties* — When lightweight aggregate concrete is used with prestressing, it must possess two important properties; the aggregates must be of high quality, and the concrete mix must have high strength. All the properties of lightweight aggregate concrete are affected to some extent by the moisture conditions of the concrete. The lightweight aggregates in general use are expanded shales, clays, and slates.

The following is a summary of the properties of prestressed lightweight concrete:

Unit weight — The range is between 100 to 120 lb per cu ft.

Compressive Strength — Only high strength concrete can be used with prestressing. In general, the commercial range of strengths is between 4000 and 6000 psi (27.58 and 41.36 MPa).

Modulus of Elasticity — An approximate formula for evaluating the modulus of elasticity of lightweight aggregate concrete in highstrength prestressed applications can be achieved by a modification of the formula listed in Section 8.5 of ACI 318-77.[80]

The above formula relates E_c values to the strength and unit weight of the concrete. In general, the ACI formula for evaluating E_c tends to overestimate E_c values at high concrete strengths.

When accurate values of E_c are required, it is suggested that either (1) a laboratory test or (2) the following modified formula be used:

$E_c = w_c^{1.5}\, C\sqrt{f_c'}$

where C is a coefficient depending upon the strength of the concrete and the other symbols are the same as those used in the ACI Code formula.[79]

$C = 31$ when $f_c' = 5000$ psi ($C = .040$ when $f_c' = 34.47$ MPa)

$C = 29$ when $f_c' = 6000$ psi ($C = .038$ when $f_c' = 41.36$ MPa)

Combined loss of prestress — This is about 110 to 115 percent of the total losses for normal weight concrete when both are subjected to normal curing; 124 percent of the total losses for normal weight concrete when both are subjected to steam curing.

Steam curing reduces the total prestress loss by 30 to 40 percent compared with normal curing.

Thermal insulation — The much greater thermal insulation of lightweight aggregate concrete has a decided effect on prestressing applications, because of the following factors:

(a) Greater temperature differential in service between the side exposed to sun and the inside may cause greater camber;
(b) Better response to steam curing;
(c) Greater suitability for winter concreting;
(d) Better fire resistance.

Dynamic, shock, vibration and seismic resistance — Prestressed lightweight concrete appears at least as good as normal weight concrete and might even be better due to its greater resilience and lower modulus of elasticity.

Cover Requirements — Permeability stress crack spacing and stress crack width of lightweight concrete are essentially the same as for normal weight concrete and require no change to cover requirements over reinforcement. Where fire requirements dictate the cover requirements, the insulating effects developed by the lower density, as well as the fire stability offered by a pre-burned aggregate may be used to considerable advantage.

5.13 — Thermal design considerations

In concrete elements exposed to environmental conditions, the choice of lightweight concrete will provide several distinct advantages over natural aggregate concrete.[90,91,92] These physical properties covered in detail in Chapter 4 are:

- The lower conductivity provides a thermal inertia that lengthens the time for exposed members to reach any steady state temperature.
- Due to this resistance, the effective interior temperature change will be smaller under transient temperature conditions. This time lag will moderate the solar build-up and nightly cooling effects.
- The lower coefficient of linear thermal expansion that is developed in the concrete due to the contribution of the lower coefficient of thermal expansion of the lightweight aggregate itself is a fundamental design consideration in exposed members. The expansion and contraction of exposed columns of tall buildings induces shearing forces and bending moments into floor frames that are connected to interior members that are subject to unchanging interior structural members. The architectural decision to locate glass window lines must of necessity take into account the conductivity and the expansion of coefficients of the exposed concretes.
- The lower modulus of expansion will develop lower stress changes in members exposed to thermal strains.

A comparative thermal investigation[92] studying the shortening developed by the average temperature of an exposed column restrained by the interior frame demonstrated the fact that the axial shortening effects were about 30 percent smaller for structural lightweight concrete and the stresses due to restrained bowing were about 35 percent less with structural lightweight concrete. The analysis, conducted on a 20 story concrete frame, used the following assumptions:

	Normal weight	Structural lightweight concrete
Thermal conductivity	12.0	5.0
Coefficient of linear thermal expansion	5.5×10^{-6}	4.5×10^{-6}
Modulus of elasticity (4.0^{ksi}) (27.58 MPa)	3.6×10^6 (24,840 MPa)	2.5×10^6 (17,250 MPa)

For an exact structural analysis use physical property data on local aggregates obtained from lightweight and natural aggregate suppliers.

Numerous practical examples demonstrating Isotherms and average temperatures developed in both lightweight and normal weight concrete exposed columns are fully shown[91] including the practical considerations of how the thermal inertia of structural lightweight concrete serves to minimize condensation. With concrete frame buildings reaching for higher heights the structural approach to controlling the temperature movements require an exact understanding of the contribution of the superior thermal response of structural lightweight concrete.

5.14 — Seismic design

Structural lightweight concrete is particularly adaptable to seismic design and construction because of the significant reduction in dead weight. A large number of multistory buildings as well as bridge structures have effectively utilized lightweight concrete in areas subject to earthquakes principally along the West Coast of the U.S. and in those countries bordering the Pacific Ocean Rim.

The lateral or horizontal forces acting upon a structure during earthquake motions are directly proportional to the inertia or weight of that structure. These lateral forces may be calculated by recognized formulas and are applied with the other load factors.

5.15 — Specifications

Lightweight concrete may be specified and proportioned on the basis of laboratory trial batches or on field experience with the materials to be employed. Most structural lightweight aggregate suppliers have mix proportioning information available for their material, and many producers provide field control and technical service to assure that the quality of concrete specified will be used.

The average strength requirements for lightweight concrete do not differ from those for normal weight concretes for the same degree of field control.

It should be observed that 28-day compressive strength tests are based on the methods of ASTM C39 which requires that the test cylinders be continuously moist-cured. One reason for confusion on this point with lightweight concrete is that ASTM C330 specifies that tests for the 28-day compressive strength to determine the concrete-making properties of a lightweight aggregate be done on test cylinders that are air dried for the final 21 days at 50 percent relative humidity. Since there is some slight improvement in apparent compressive strength when the specimens are tested air-dried, the standard test method leads to conservative test values. Cylinders continuously moist-cured should not be weighed and measured and used to determine the 28-day density of the concrete.

Lightweight aggregate concrete that, after curing, will be exposed to freezing shall have a specified compressive strength f_c' of at least 3000 psi (20.68 MPa) and have entrained air in accordance with ACI 318.77.[80]

Lightweight aggregate concrete that is intended to be watertight shall have a specified compressive strength f_c' of at least 3750 psi (25.84 MPa) for exposure to fresh water and 4000 psi (27.58 MPa) for exposure to sea water.

Lightweight aggregate concrete that will be exposed to injurious concentrations of sulfate-containing solutions shall be made with sulfate-resisting cement and have a specified compressive strength f_c' of at least 3750 psi (25.84 MPa).

Splitting tensile strength tests shall not be used as a basis for field acceptance of lightweight aggregate concrete.

The analysis of the load-carrying capacity of a lightweight concrete structure either by cores or load tests shall be the same as for normal weight concrete.

In general, most structural lightweight aggregate suppliers have suggested specifications pertaining to their material.

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This report was submitted to letter ballot of the committee which consisted of 25 members; 21 members returned affirmative ballots, 4 ballots were not returned.

ACI Committee 213

Guide for Structural Lightweight Aggregate Concrete

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ANSI/ACI 214-77

ACI Standard

Recommended Practice for Evaluation of Strength Test Results of Concrete (ACI 214-77)*

Reported by ACI Committee 214

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Statistical procedures provide valuable tools for assessing results of strength tests, and such an approach is also of value in refining design criteria and specifications. The report discusses briefly the numerous variations that occur in the strength of concrete and presents statistical procedures which are useful in interpreting these variations.

Keywords: coefficient of variation; **compression tests**; compressive strength; concrete construction; **concretes**; cylinders; **evaluation**; **quality control**; sampling; standard deviation; **statistical analysis**; variations.

CONTENTS

*Adopted as a standard of the American Concrete Institute in August 1977, to supersede ACI 214-65, in accordance with the Institute's standardization procedure.
†Chairman during development of the revision.

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CHAPTER I—INTRODUCTION

The purposes of strength tests of concrete are to determine compliance with a strength specification and to measure the variability of concrete. Concrete, being a hardened mass of heterogeneous materials, is subject to the influence of numerous variables. Characteristics of each of the ingredients of concrete, depending on their variability, may cause variations in strength of concrete. Variations may also be introduced by practices used in proportioning, mixing, transporting, placing, and curing. In addition to the variations which exist in concrete itself, test strength variations will also be introduced by the fabrication, testing, and treatment of test specimens. Variations in the strength of concrete must be accepted, but concrete of adequate quality can be produced with confidence if proper control is maintained, test results are properly interpreted, and their limitations are considered.

Proper control is achieved by the use of satisfactory materials, correct batching and mixing of these materials, correct batching and mixing of sired quality, and good practices in transporting, placing, curing, and testing. Although the complex nature of concrete precludes complete homogeneity, excessive variation of concrete strength signifies inadequate concrete control. Improvement in control may permit a reduction in the cost of concrete since the average strength can be brought closer to specification requirements.

Strength is not necessarily the most critical factor in proportioning concrete mixes since other factors, such as durability, may impose lower water-cement ratios than are required to meet strength requirements. In such cases, strength will of necessity be in excess of structural demands. Nevertheless, strength tests are valuable in such circumstances since, with established mix proportions, variations in strength are indicative of variations in other properties.

Test specimens indicate the potential rather than the actual strength of the concrete in a structure. To be meaningful, conclusions on strength of concrete must be derived from a pattern of tests from which the characteristics of the concrete can be estimated with reasonable accuracy. Insufficient tests will result in unreliable conclusions.

Statistical procedures provide tools of considerable value in evaluating results of strength tests and information derived from such procedures is also of value in refining design criteria and specifications. This report briefly discusses variations that occur in the strength of concrete, and presents statistical procedures that are useful in the interpretation of these variations with respect to required criteria and specifications. For these statistical procedures to be valid, the data must be derived from samples obtained by means of a random sampling plan designed to reduce the possibility that choice will be exercised by the sampler. "Random sampling" means that each possible sample has an equal chance of being selected. To insure this condition, the choice must be made by some objective mechanism such as a table of random numbers. If sample batches are selected by the sampler on the basis of his own judgment, biases are likely to be introduced that will invalidate results analyzed by the procedures presented here. Reference 1 contains a discussion of random sampling and a useful short table of random numbers.

Additional information on the meaning and use of this recommended practice is given in *Realism in the Application of ACI Standard 214-65.*[2] This volume is a compilation of information on ACI 214-65 that was presented at a symposium held at Buffalo, N. Y., in 1971. In addition to the papers from the symposium, it includes reprints of some pertinent papers that were published earlier in the ACI JOURNAL, and of discussion that resulted from them. Although the information given was based on ACI 214-65, most of it is still relevant. An additional source of material on evaluation of strength tests is ACI Bibliography No. 2, published in 1960.[3]

CHAPTER 2—VARIATIONS IN STRENGTH

2.1—General

The magnitude of variations in the strength of concrete test specimens depends on how well the materials, concrete manufacture, and testing are controlled. Differences in strength can be traced to two fundamentally different sources as shown in Table 2.1: (a) differences in strength-producing properties of the concrete mixture and ingredients, and (b) apparent differences in strength caused by variations inherent in the testing.

2.2—Properties of concrete

It is well established that strength is governed to a large extent by the water-cement ratio. The

TABLE 2.1—PRINCIPAL SOURCES OF STRENGTH VARIATION

Variations in the properties of concrete	Discrepancies in testing methods
Changes in water-cement ratio: Poor control of water Excessive variation of moisture in aggregate Retempering	Improper sampling procedures
Variations in water requirement: Aggregate grading, absorption, particle shape Cement and admixture properties Air content Delivery time and temperature	Variations due to fabrication techniques Handling and curing of newly made cylinders Poor quality molds
Variations in characteristics and proportions of ingredients: Aggregates Cement Pozzolans Admixtures	Changes in curing: Temperature variation Variable moisture Delays in bringing cylinders to the laboratory
Variations in transporting, placing, and compaction Variations in temperature and curing	Poor testing procedures: Cylinder capping Compression tests

first criterion for producing concrete of constant strength, therefore, is a constant water-cement ratio. Since the quantity of cement and added water can be measured accurately, the problem of maintaining a constant water-cement ratio is primarily one of correcting for the variable quantity of free moisture in aggregates.

The homogeneity of concrete is influenced by the variability of the aggregates, cement, and admixtures used, since each will contribute to variations in the concrete strength. The temperature of fresh concrete influences the amount of water needed to achieve the proper consistency and consequently contributes to strength variation. Construction practices may cause variations in strength due to inadequate mixing, poor compaction, delays, and improper curing. Not all of these are reflected in specimens fabricated and stored under standard conditions.

The use of admixtures adds another factor since each admixture adds another variable to concrete. The batching of accelerators, retarders, pozzolans, and air-entraining agents must be carefully controlled.

2.3—Testing methods

Concrete tests may or may not include all the variations in strength of concrete in place depending on what variables have been introduced after test specimens were made. On the other hand, discrepancies in sampling, fabrication curing, and testing of specimens may cause indications of variations in strength which do not exist in the concrete in the structure. The project is unnecessarily penalized when variations from this source are excessive. Good testing methods will reduce these variations, and standard testing procedures such as those described in ASTM standards should be followed without deviation.

The importance of using accurate testing machines and producing thin, high-strength, plane, parallel caps should need no emphasis since test results can be no more accurate than the equipment and procedures used. *Uniform test results are not necessarily accurate test results.* Laboratory equipment and procedures should be calibrated and checked periodically.

CHAPTER 3—ANALYSIS OF STRENGTH DATA

3.1—Notation

d_2 and $1/d_2$ = factors for computing within-test standard deviation from average range

f_{cr} = required average strength to assure that no more than the permissible proportion of tests will fall below specified strength

f_c' = specified strength

n = number of tests

R = range

$\overline{R}_m$ = maximum for average range used in control charts for moving average for range

$\overline{R}$ = average range

σ = standard deviation

σ_1 = within-test standard deviation

σ_2 = batch-to-batch standard deviation

t = a constant multiplier for standard deviation (σ) that depends on the number of tests expected to fall below f_c'

V = coefficient of variation

V_1 = within-test coefficient of variation

X_i = an individual test result

$\overline{X}$ = average of test results

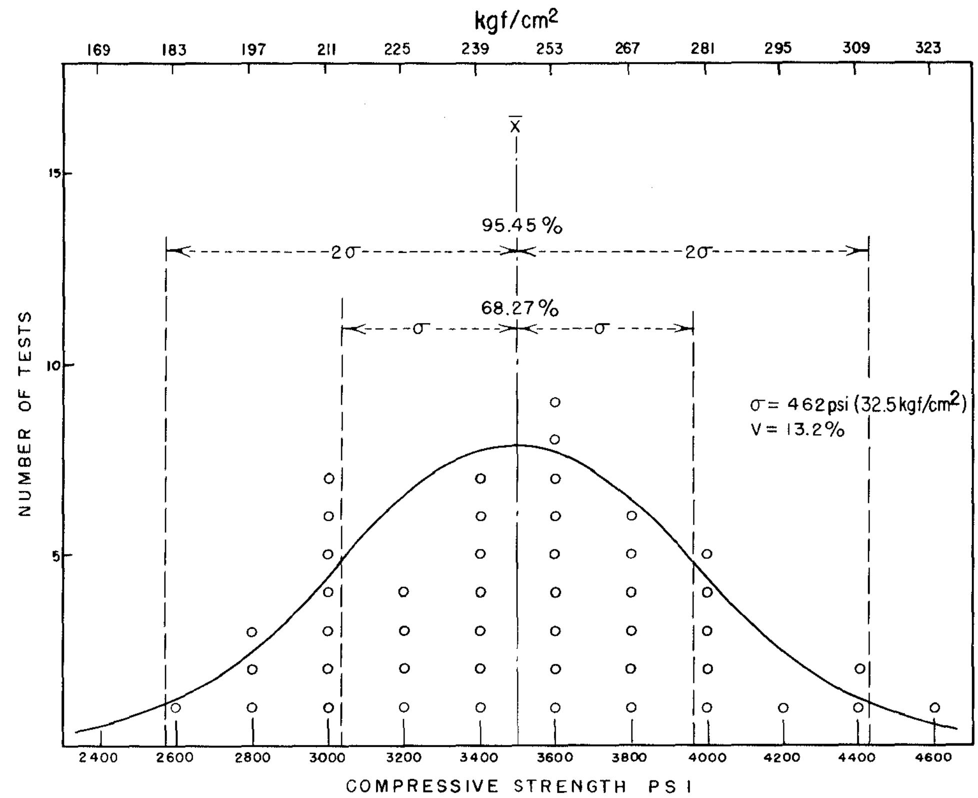

Fig. 3.3(a)—Frequency distribution of strength data and corresponding normal distribution

3.2—General

To obtain maximum information, a sufficient number of tests should be made to indicate the variation in the concrete produced and to permit appropriate statistical procedures to be used in interpreting the test results. Statistical procedures provide the best basis for determining from such results the potential quality and strength of the concrete and for expressing results in the most useful form.

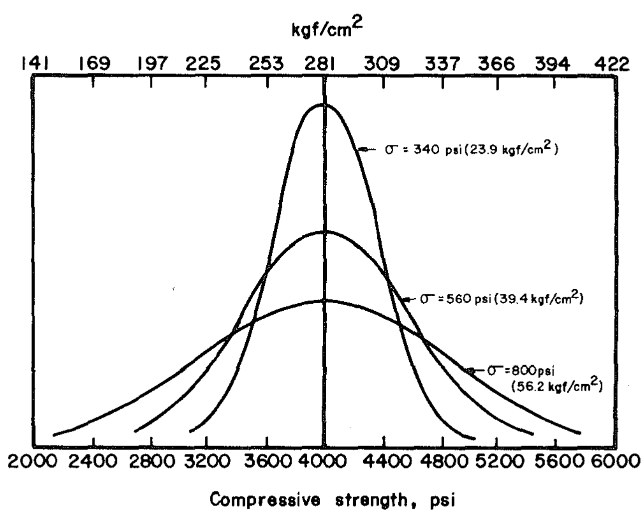

Fig. 3.3(b)—Normal frequency curves for different standard deviations

3.3—Statistical functions

The strength of concrete test specimens on controlled projects can be assumed to fall into a pattern similar to the normal frequency distribution curve illustrated in Fig. 3.3 (a). Where there is good control, the strength values will be bunched close to the average, and the curve will be tall and narrow. As the variations in strength increase, the values spread and the curve becomes low and elongated, as illustrated by the idealized curves shown in Fig. 3.3 (b). Because the characteristics of such curves can be defined mathematically, certain useful functions of the strength can be calculated as follows:

3.3.1 *Average,* $\overline{X}$—The average strength of all individual tests

$$\overline{X} = \frac{X_1 + X_2 + X_3 + \ldots + X_n}{n} \qquad (3\text{-}1)$$

Where X_1, X_2, X_3 . . . X_n are the strength results of individual tests and n is the total number of tests made. A test is defined as the average strength of all specimens of the same age fabricated from a sample taken from a single batch of concrete.

3.3.2 *Standard deviation,* σ—The most generally recognized measure of dispersion is the root-mean-square deviation of the strengths from their average. This statistic is known as the standard deviation and may be considered to be the radius of gyration about the line of symmetry of the area under the curve of the frequency distribution of strength data, such as that shown in Fig. 3.3 (a). The best estimate of σ, based on a finite amount of data, is obtained by Eq. (3-2), or by its algebraic equivalent, Eq. (3-2a). The latter equation is preferable for computation purposes, because it is not only simpler and more adaptable to desk calculators, but it avoids the possibility of trouble due to rounding errors.

$$\sigma = \{[(X_1 - X)^2 + (X_2 - X)^2 + \ldots + (X_n - X)^2]/n - 1\}^{1/2} \qquad (3\text{-}2)$$

or

$$\sigma = \sqrt{\frac{\Sigma X_i^2 - \frac{(\Sigma X_i)^2}{n}}{n-1}} \qquad (3\text{-}2a)$$

3.3.3 *Coefficient of variation, V*—The standard deviation expressed as a percentage of the average strength is called the coefficient of variation:

$$V = \frac{\sigma}{\overline{X}} \times 100 \qquad (3\text{-}3)$$

3.3.4 *Range, R*—Range is the statistic found by subtracting the lowest of a group of numbers from the highest one in the group. The within-test range is found by subtracting the lowest of the group of cylinder strengths averaged to produce a test from the highest of the group. The within-test range is useful in computing the within-test standard deviation discussed in the following section.

3.4—Strength variations

As mentioned previously, variations in results of strength tests can be traced to two different sources: (a) variations in testing methods and (b) properties of the concrete mixture and ingredients. It is possible by analysis of variance to compute the variations attributable to each source.

3.4.1 *Within-test variation* — The variation in strength of concrete within a single test is found by computing the variation of a group of cylinders fabricated from a sample of concrete taken from a given batch. It is reasonable to assume that a test sample of concrete is homogeneous and any variation between companion cylinders fabricated from a given sample is caused by fabricating, curing, and testing variations.

A single batch of concrete, however, provides insufficient data for statistical analysis and companion cylinders from at least ten batches of concrete are required to establish reliable values for $\overline{R}$. The within-test standard deviation and coefficient of variation can be conveniently computed as follows:

TABLE 3.4.1—FACTORS FOR COMPUTING WITHIN-TEST STANDARD DEVIATION*

Number of specimens	d_2	$1/d_2$
2	1.128	0.8865
3	1.693	0.5907
4	2.059	0.4857
5	2.326	0.4299
6	2.534	0.3946
7	2.704	0.3698
8	2.847	0.3512
9	2.970	0.3367
10	3.078	0.3249

*From Table B2, *ASTM Manual on Quality Control of Materials*, Reference 4.

$$\sigma_1 = \frac{1}{d_2}\overline{R} \qquad (3\text{-}4)$$

$$V_1 = \frac{\sigma_1}{\overline{X}} \times 100 \qquad (3\text{-}5)$$

where

σ_1 = within-test standard deviation

$1/d_2$ = a constant depending on the number of cylinders averaged to produce a test (Table 3.4.1)

$\overline{R}$ = average range within groups of companion cylinders

V_1 = within-test coefficient of variation

$\overline{X}$ = average strength

3.4.2 *Batch-to-batch variations*—These variations reflect differences in strength which can be attributed to variations in

(a) Characteristics and properties of the ingredients

(b) Batching, mixing, and sampling

(c) Testing that has not been detected from companion cylinders since these tend to be treated more alike than cylinders tested at different times

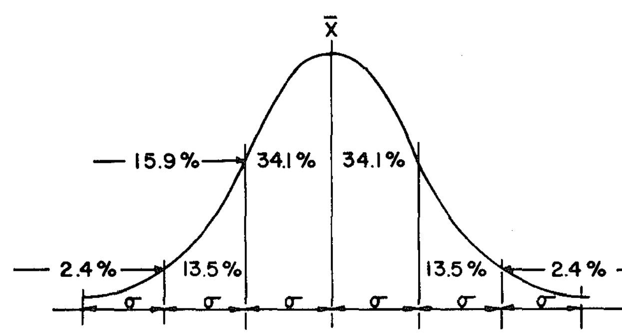

Fig. 3.4.2(a)—Approximate division of the area under the normal frequency distribution curve

The batch-to-batch and within-test sources of variation are related to the overall variation [Eq. (3-3)] by the following expression:

$$\sigma^2 = \sigma_1^2 + \sigma_2^2 \qquad (3\text{-}6)$$

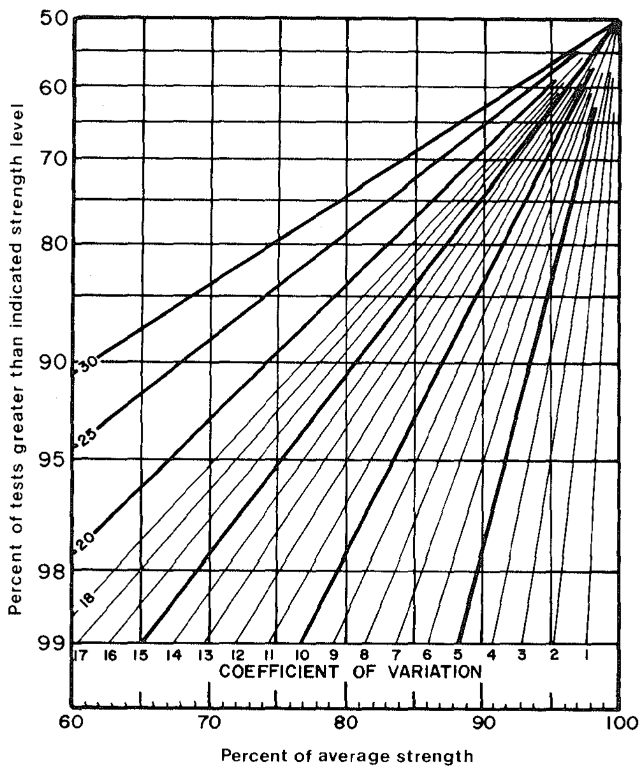

Fig. 3.4.2(b)—Cumulative distribution curves for different coefficients of variation

where

σ = overall standard deviation

σ_1 = within-test standard deviation

σ_2 = batch-to-batch standard deviation

Once these parameters have been computed, and with the assumption that the results follow a normal frequency distribution curve, a large amount of information about the test results becomes known. Fig. 3.4.2(a) indicates an approximate division of the area under the normal frequency distribution curve. For example, approximately 68 percent of the area (equivalent to 68 percent of the test results) lies within $\pm 1\sigma$ of the average, 95 percent within $\pm 2\sigma$, etc. This permits an estimate to be made of the portion of

TABLE 3.4.2—EXPECTED PERCENTAGES OF TESTS LOWER THAN f_c' WHERE $\overline{X}$ EXCEEDS f_c' BY THE AMOUNT SHOWN

Average strength, $\overline{X}$	Expected percentage of low tests	Average strength, $\overline{X}$	Expected percentage of low tests
$f_c' + 0.10\sigma$	46.0	$f_c' + 1.6\sigma$	5.5
$f_c' + 0.20\sigma$	42.1	$f_c' + 1.7\sigma$	4.5
$f_c' + 0.30\sigma$	38.2	$f_c' + 1.8\sigma$	3.6
$f_c' + 0.40\sigma$	34.5	$f_c' + 1.9\sigma$	2.9
$f_c' + 0.50\sigma$	30.9	$f_c' + 2.0\sigma$	2.3
$f_c' + 0.60\sigma$	27.4	$f_c' + 2.1\sigma$	1.8
$f_c' + 0.70\sigma$	24.2	$f_c' + 2.2\sigma$	1.4
$f_c' + 0.80\sigma$	21.2	$f_c' + 2.3\sigma$	1.1
$f_c' + 0.90\sigma$	18.4	$f_c' + 2.4\sigma$	0.8
$f_c' + \sigma$	15.9	$f_c' + 2.5\sigma$	0.6
$f_c' + 1.10\sigma$	13.6	$f_c' + 2.6\sigma$	0.45
$f_c' + 1.20\sigma$	11.5	$f_c' + 2.7\sigma$	0.35
$f_c' + 1.30\sigma$	9.7	$f_c' + 2.8\sigma$	0.25
$f_c' + 1.40\sigma$	8.1	$f_c' + 2.9\sigma$	0.19
$f_c' + 1.50\sigma$	6.7	$f_c' + 3.0\sigma$	0.13

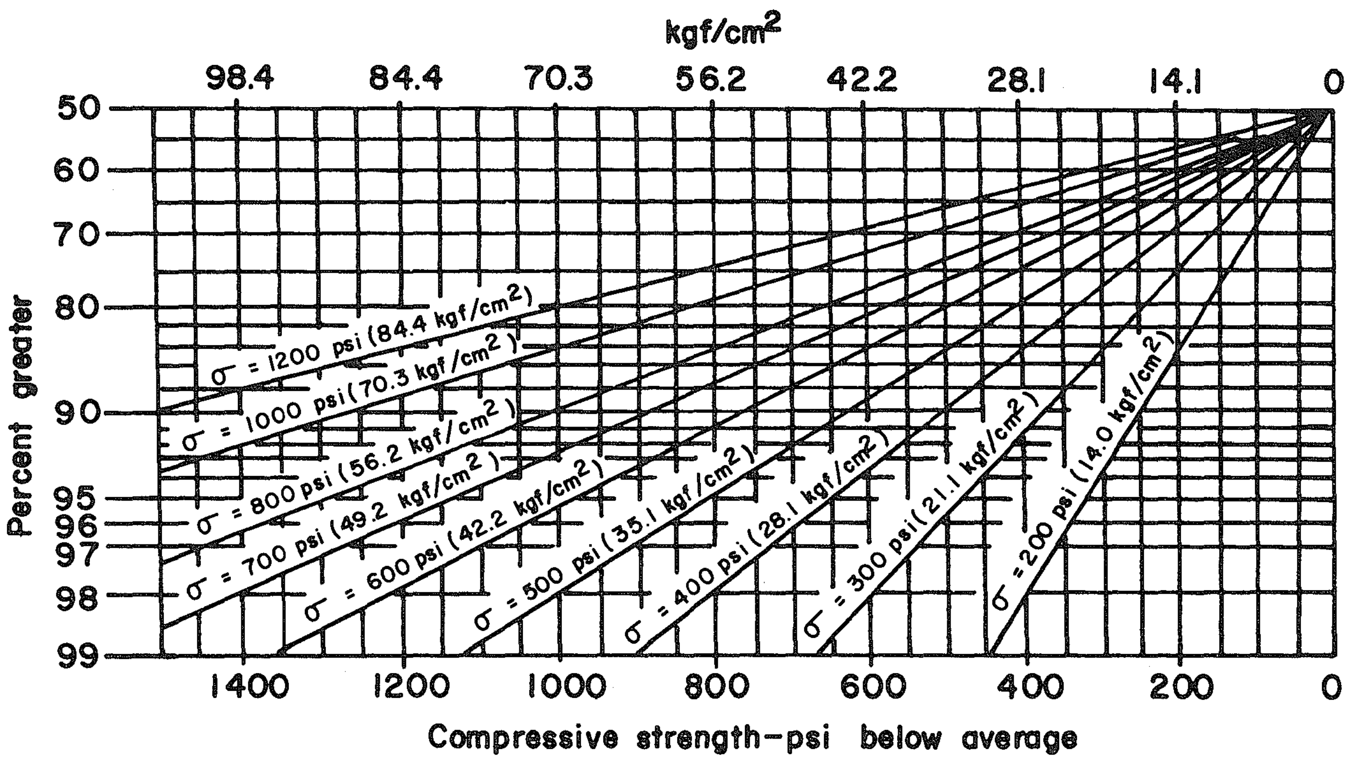

Fig. 3.4.2(c)—Cumulative distribution curves for different standard deviations

TABLE 3.5—STANDARDS OF CONCRETE CONTROL

Overall variation					
Class of operation	Standard deviation for different control standards, psi (kgf/cm²)				
	Excellent	Very good	Good	Fair	Poor
General construction testing	below 400 (28.1)	400 to 500 (28.1) (35.2)	500 to 600 (35.2) (42.2)	600 to 700 (42.2) (49.2)	above 700 (49.2)
Laboratory trial batches	below 200 (14.1)	200 to 250 (14.1) (17.6)	250 to 300 (17.6) (21.1)	300 to 350 (21.1) (24.6)	above 350 (24.6)

Within-test variation					
Class of operation	Coefficient of variation for different control standards, percent				
	Excellent	Very good	Good	Fair	Poor
Field control testing	below 3.0	3.0 to 4.0	4.0 to 5.0	5.0 to 6.0	above 6.0
Laboratory trial batches	below 2.0	2.0 to 3.0	3.0 to 4.0	4.0 to 5.0	above 5.0

the test results expected to fall within given multiples of σ of the average or of any other specific value. Table 3.4.2 has been adapted from the normal probability integral of the theoretical normal frequency distribution curve and shows the probability of tests falling below f_c' in terms of the average strength of the mix $\overline{X} = f_{cr} = (f_c' + t\sigma)$. Cumulative distribution curves can also be plotted by accumulating the number of tests below any given strength expressed as a percentage of the average strength for different coefficients of variation or standard deviations. Fig. 3.4.2(b) and 3.4.2(c) present such information.

In these figures, the ordinate indicates the percent of the population of strength values which may be expected to exceed the strength indicated by any abscissa value for a selected coefficient of variation or standard deviation.

3.5—Standards of control

The decision as to whether the standard deviation or the coefficient of variation is the appropriate measure of dispersion to use in any given situation depends on which of the two measures is the more nearly constant over the range of strengths characteristic of the particular situation. Present information indicates that the standard deviation remains more nearly constant particularly at strengths over 3000 psi (211 kgf/cm²). For within-test variations the coefficient of variation is considered to be more applicable (see References 5-10).

Table 3.5 shows the variability that can be expected for compressive strength tests on projects subject to different degrees of control. These values are not applicable to other strength tests.

CHAPTER 4—CRITERIA

4.1—General

The strength of control cylinders is generally the only tangible evidence of the quality of concrete used in constructing a structure. Because of the possible disparity between the strength of test cylinders and the load-carrying capacity of a structure it is unwise to place any reliance on inadequate strength data.

The number of tests lower than the desired strength is more important in computing the load-carrying capacity of concrete structures than the average strength obtained. It is impractical, however, to specify a minimum strength since there is always the possibility of even lower strengths, even when control is good. It is also recognized that the cylinders may not accurately represent the concrete in each portion of the structure. Factors of safety are provided in design equations which allow for deviations from specified strengths without jeopardizing the safety of the structure. These have been evolved on the basis of construction practices, design procedures, and quality control techniques used by the construction industry. It should also be remembered that for a given mean strength, if a small percentage

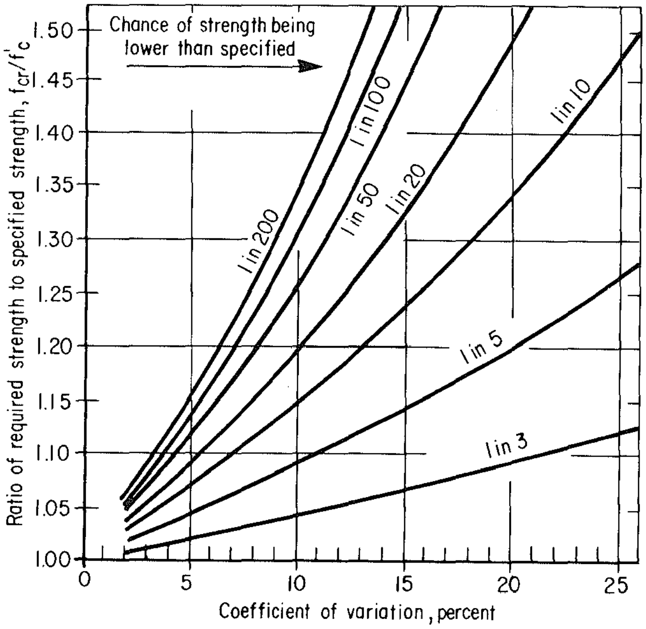

Fig. 4.1(a)—Ratio of required average strength f_{cr} to specified strength f_c' for various coefficients of variation and chances of falling below specified strength

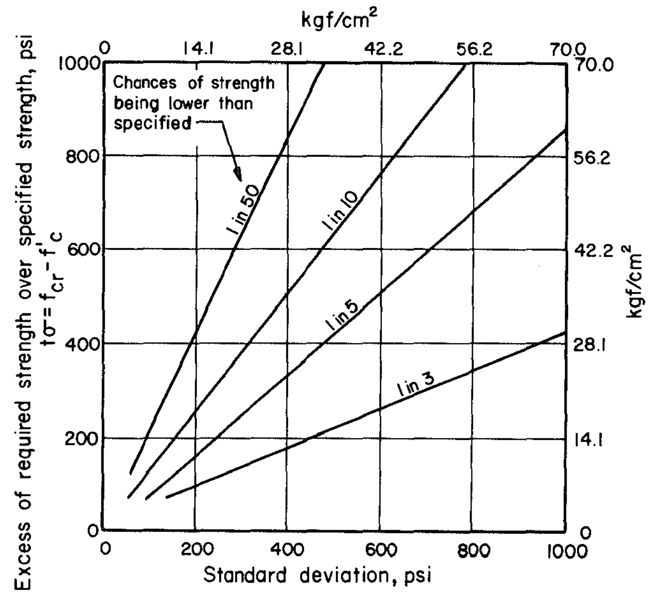

Fig. 4.1(b)—Excess of required average strength f_{cr} to specified strength f_c' for various standard deviations and chances of falling below specified strength

of the test results fall below the design strength, a corresponding large percentage of the test results will be greater than the design strength with an equally large probability of being located in a critical area. The consequences of a localized zone of low-strength concrete in a structure depend on many factors; included are the probability of early overload, the location and magnitude of the low-quality zone in the structural unit, the degree of reliance placed on strength in design, the initial cause of the low strength, and the consequences, economic and otherwise, of structural failure.

The final criterion which allows for a certain probability of tests falling below f_c' used in design is a designer's decision based on his intimate knowledge of the conditions that are likely to prevail. "Building Code Requirements for Reinforced Concrete (ACI 318-71)," provides guidelines in this regard, as do other building codes and specifications.

To satisfy strength performance requirements expressed in this fashion the average strength of concrete must be in excess of f_c', the design strength. The amount of excess strength depends on the expected variability of test results as expressed by a coefficient of variation or standard deviation, and on the allowable proportion of low tests.

Strength data for determining the standard deviation or coefficient of variation should represent a group of at least 30 consecutive tests made on concrete produced under conditions similar to those to be expected on the project. The requirement for 30 consecutive strength tests will be considered to have been complied with if the tests represent either a group of 30 consecutive batches of the same class of concrete or the statistical average for two groups totalling 30 or more batches. "Similar" conditions will be difficult to define and can be best documented by collecting several groups of 30 or more tests. In general, changes in materials and procedures will have a larger effect on the average strength level than on the standard deviation or coefficient of variation. Significant changes generally include changes in type and brand of portland cement, admixtures, source of aggregates, mix proportions, batching, mixing, delivery, or testing. The data should represent concrete produced to meet a specified strength close to that specified for the proposed work, since the standard deviation may vary as the average strength varies. The required average strength f_{cr} for any design can be computed from Eq. (4-1) or (4-1a), (Table 3.4.2), or approximated from Fig. 4.1(a) or 4.1(b), depending on whether the coefficient of variation or standard deviation is used.

$$f_{cr} = \frac{f_c'}{(1 - tV)} \tag{4-1}$$

$$f_{cr} = f_c' + t\sigma \tag{4-1a}$$

where

f_{cr} = required average strength
f_c' = design strength specified
t = a constant depending upon the proportion of tests that may fall below f_c' (Table 4.1)
V = forecast value of the coefficient of variation expressed as a fraction
σ = forecast value of the standard deviation

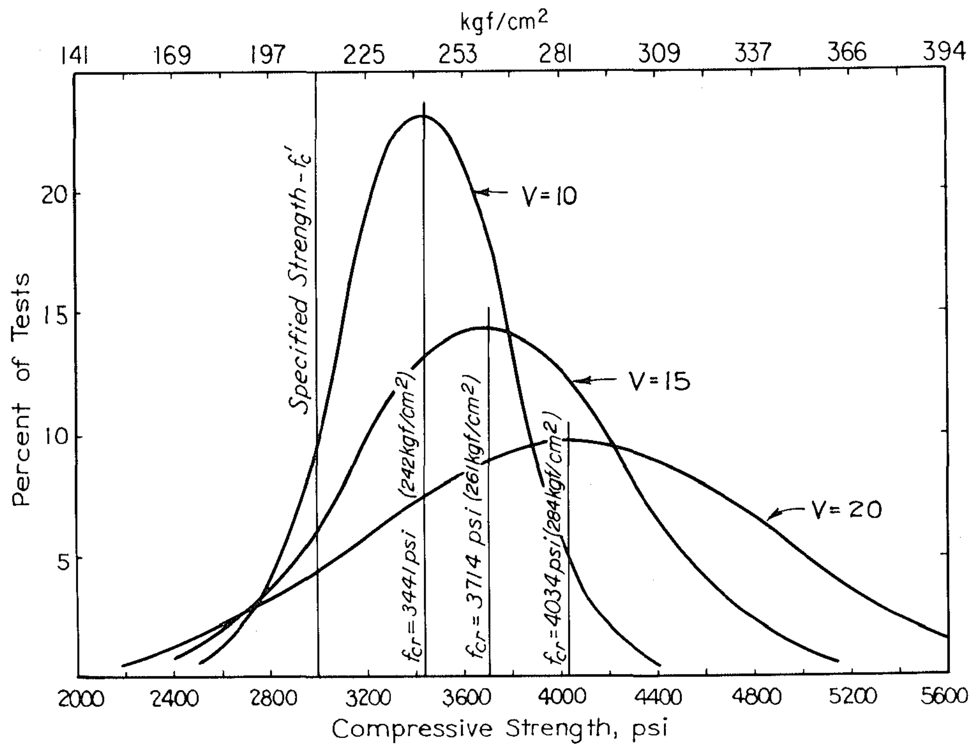

Fig. 4.1(c)—Normal frequency curves for coefficients of variation of 10, 15, and 20 percent

Whenever the average of a certain number of tests n is involved in the specification, Eq. (4-1) is modified as follows:

$$f_{cr} = \frac{f_c'}{1 - \frac{tV}{\sqrt{n}}} \qquad \text{(4-1b)}$$

and

$$f_{cr} = f_c' + \frac{t\sigma}{\sqrt{n}} \qquad \text{(4-1c)}$$

Fig. 4.1 (c) demonstrates that as the variability increases f_{cr} must increase and thereby illustrates the economic value of good control.

The requirement of at least 30 test results mentioned previously is based on the fact that 25 to 30 randomly selected test results from a normally distributed population provide estimates of the population average and standard deviation that can be used as the population values. If only a small number of results is available on which to base estimates, then the values, especially for standard deviation, are unreliable, and there is no way in which f_{cr} can be determined so that a specific percentage of future tests will be above f_c', assuming that the present test results are the only information available.

If previous information exists for concrete from the same plant meeting the similarity requirements described above, that information may be used in deciding on a trial value of σ to be used in determining the target f_{cr}.

TABLE 4.1—VALUES OF t

Percentages of tests falling within the limits $\overline{X} \pm t\sigma$	Chances of falling below lower limit	t
40	3 in 10	0.52
50	2.5 in 10	0.67
60	2 in 10	0.84
68.27	1 in 6.3	1.00
70	1.5 in 10	1.04
80	1 in 10	1.28
90	1 in 20	1.65
95	1 in 40	1.96
95.45	1 in 44	2.00
98	1 in 100	2.33
99	1 in 200	2.58
99.73	1 in 741	3.00

For small jobs that are just getting started, where no prior information is available, the concrete should be designed to produce an average strength f_{cr} at least 1200 psi (84.4 kgf/cm^2) greater than f_c'. As the job progresses and more strength tests become available, all the strength tests can be analyzed together to give a more reliable estimate of the standard deviation, and Eq. (4-1), (4-1a), (4-1b), and (4-1c) can be used to calculate a less conservative f_{cr}.

4.2—Criteria for strength requirements

The amount by which the average strength of a concrete mix f_{cr} should exceed f_c' depends on the criteria used in the specifications for a particular project. The following are examples of calculations that would have to be made to select the design strengths of a mix that will meet the requirements of a particular code or specification.

4.2.1 *Criterion No. 1*—A stated maximum proportion of random individual strength tests that will be permitted to fall below f_c' on the average.

ASTM C 94-74 uses a similar approach. For concrete in structures designed by the ultimate strength method, ASTM recommends that not more than 10 percent of the strength tests have values less than the specified strength f_c'.

As an example, consider the situation where no more than 1 in 10 random individual strengths will be permitted to be below an f_c' of 4000 psi (281 kgf/cm^2).

Standard deviation method

Consider very good quality control as indicated by a standard deviation of 450 psi (31.7 kgf/cm^2). Using Eq. (4-1a) and Table 4.1, we have

$$\begin{aligned} f_{cr} &= f_c' + t\sigma \\ &= 4000 + 1.28 \times 450 \\ &= 4580 \text{ psi } (322 \text{ kgf/cm}^2) \end{aligned}$$

As a result, for a structural design strength f_c' of 4000 psi (281 kgf/cm^2), the concrete mixture should be proportioned for an average strength of not less than 4580 psi (322 kgf/cm^2). Note that the coefficient of variation is (450/4580) $\times$ 100 = 9.8 percent.

Coefficient of variation method

Consider good quality control as indicated by a coefficient of variation of 10 percent. Using Eq. (4-1) and Table 4.1, we have

$$f_{cr} = \frac{f_c'}{1 - tV}$$

$$\begin{aligned} f_{cr} &= \frac{f_c'}{1 - 1.28\,(0.10)} \\ &= 1.15\, f_c' \text{ [see also Fig. 4.1 (a)]} \\ &= 4600 \text{ psi } (324 \text{ kgf/cm}^2) \end{aligned}$$

Using this approach and this data the concrete mixture should be proportioned for an average strength of not less than 4600 psi (324 kgf/cm^2).

4.2.2 *Criterion No. 2*—A certain probability that an average of n consecutive strength tests will be below f_c'.

ACI 318-71 suggests that after sufficient test data become available from a project, the frequency of occurrence of averages of three consecutive tests below f_c' should not exceed 1 in 100.

As an example, consider the situation where no more than 1 in 100 of averages of three consecutive strength tests will be permitted to be below an f_c' of 4000 psi (281 kgf/cm^2).

Standard deviation method

Consider a standard deviation of 750 psi (53 kgf/cm^2). Using Eq. (4-1c) and Table 4.1, we have

$$\begin{aligned} f_{cr} &= f_c' + \frac{t\sigma}{\sqrt{n}} \\ &= 4000 \text{ psi} + \frac{2.33\,(750)}{\sqrt{3}} \\ &= 5000 \text{ psi } (351 \text{ kgf/cm}^2) \end{aligned}$$

As a result, for a structural design strength f_c' of 4000 psi (281 kgf/cm^2), the concrete mixture should be proportioned for an average strength of not less than 5000 psi (351 kgf/cm^2).

Coefficient of variation method

Considering a coefficient of variation of 15 percent and using Eq. (4-1b) and Table 4.1, we have

$$\begin{aligned} f_{cr} &= \frac{f_c'}{1 - \dfrac{tV}{\sqrt{n}}} \\ &= \frac{4000}{1 - \dfrac{2.33\ (0.15)}{\sqrt{3}}} \\ &= 5000 \text{ psi } (351 \text{ kgf/cm}^2) \end{aligned}$$

Using this approach the concrete mixture should be proportioned for an average strength of not less than 5000 psi (351 kgf/cm^2).

4.2.3 *Criterion No. 3*—A certain probability that a random individual strength test will be more than a certain amount below f_c'.

This approach is also used in ACI 318-71 by stipulating that the probability of a random test result being more than 500 psi (35.1 kgf/cm^2) below f_c' should be 1 in 100.

As an example, consider a probability of 1 in 100 that a strength test will be more than 500 psi (35.1 kgf/cm^2) below an f_c' of 4000 psi (281 kgf/cm^2).

Standard deviation method

Considering a standard deviation of 750 psi (53 kgf/cm^2) and using Eq. (4-1a) and Table 4.1, we have

$$\begin{aligned} f_{cr} &= f_c' - 500 + t\sigma \\ &= 4000 - 500 + 2.33\ (750) \\ &= 5245 \text{ psi } (369 \text{ kgf/cm}^2) \end{aligned}$$

As a result the concrete mixture should be proportioned for an average strength of not less than 5245 psi (369 kgf/cm^2).

Coefficient of variation method

Using Eq. (4-1) and Table 4.1, and a coefficient of variation of 15 percent, we have

$$f_{cr} = \frac{f_c' - 500}{1 - tV}$$

$$\begin{aligned} f_{cr} &= \frac{4000 - 500}{1 - 2.33\ (0.15)} \\ &= 5390 \text{ psi } (379 \text{ kgf/cm}^2) \end{aligned}$$

Using this approach, the concrete mixture should be proportioned for an average strength of not less than 5390 psi (379 kgf/cm^2).

TABLE 4.3—EVALUATION OF CONSECUTIVE LOW STRENGTH TEST RESULTS

1	2	3	4	5
Number of consecutive tests averaged	Averages less than indicated require investigation*			Probability of averages less than f_c',† percent
	Criteria for original selection of f_{cr}			
	1 test in 10 below f_c'		1 test in 100 less than [$f_c' - 500$ psi (35.2 kgf/cm^2)]	1 test in 10 below f_c'
	For $V = 15$, percent	For given σ	For given σ	
1	$0.86f_c'$	$f_c' - 0.77\sigma$	$f_c' - 500 + 0.76\sigma$	10.0
2	$0.97f_c'$	$f_c' - 0.17\sigma$	$f_c' - 500 + 0.88\sigma$	3.5
3	$1.02f_c'$	$f_c' + 0.10\sigma$	$f_c' - 500 + 1.14\sigma$	1.3
4	$1.05f_c'$	$f_c' + 0.26\sigma$	$f_c' - 500 + 1.30\sigma$	0.5
5	$1.07f_c'$	$f_c' + 0.36\sigma$	$f_c' - 500 + 1.41\sigma$	0.2
6	$1.08f_c'$	$f_c' + 0.44\sigma$	$f_c' - 500 + 1.49\sigma$	0.1

*The probability of averages less than the levels indicated is approximately 2 percent if the population average equals f_{cr} and the standard deviation or coefficient of variation is at the level assumed.

†If the population average equals f_{cr} and the standard deviation or coefficient of variation is at the level assumed.

4.2.4 *Criterion No. 4*—A certain probability that a random individual strength test will be less than a certain percentage of f_c'.

As an example consider a probability of 1 in 100 that a strength test will be less than 85 percent of an f_c' of 4000 psi (281 kgf/cm^2).

Standard deviation method

Using Eq. (4-1a) and Table 4.1 and a standard deviation of 750 (53 kgf/cm^2), we have

$$\begin{aligned} f_{cr} &= 0.85\, f_c' + t\sigma \\ &= 0.85\ (4000) + 2.33\ (750) \\ &= 5145 \text{ psi } (361 \text{ kgf/cm}^2) \end{aligned}$$

As a result the concrete mixture should be proportioned for an average strength of not less than 5145 psi (361 kgf/cm^2).

Coefficient of variation method

Using Eq. (4-1) and Table 4.1 and a coefficient of variation of 15 percent, we have

$$\begin{aligned} f_{cr} &= \frac{0.85\, f_c'}{1 - tV} \\ &= \frac{0.85\ (4000)}{1 - 2.33\ (0.15)} \\ &= 5230 \text{ psi } (368 \text{ kgf/cm}^2) \end{aligned}$$

Using this approach, the concrete mixture should be proportioned for an average strength of not less than 5230 psi (368 kgf/cm^2).

4.3—Additional information

Table 4.3 presents additional information. The values in the body of the table in Columns 2, 3, and 4 are the strength levels below which individual tests or averages of different numbers of tests should not normally fall. These values are based on the premise that the concrete is proportioned to produce an average strength equal to f_{cr}. The values in Column 2 are theoretically correct only for concrete with a coefficient of variation of 15 percent. Those in Columns 3 and 4 apply to any known standard deviation. In either case the probability of their being exceeded when the concrete is properly controlled is only about 0.02. Thus, failure to meet the tabulated limits in a larger proportion of cases than that stated may be an indication that the current average strength is less than f_{cr} or that σ or V has increased. This could be caused by lower strength or poorer control than expected, or both. The possibility should not be overlooked that the low tests may be caused by errors in sampling or testing rather than deficiency in the concrete itself. In any case, corrective action is warranted.

Column 5 shows the probability that the average of any given number of consecutive tests will fail to equal or exceed f_c' if the concrete is proportioned to produce an average strength equal to f_{cr}. It can be seen that increasing the number of tests to be averaged increases the likelihood that f_c' will be exceeded since variations tend to balance out with an increased number of tests in a set. For enforcement purposes, it is appropriate and logical to select the number of consecutive tests to be averaged in such a way that the acceptance level is equal to f_c'. This would mean an average of three consecutive tests for concrete in which one out of ten tests would be permitted to be lower than f_c'. It should, however, be remembered that, according to the statistical theory assumed in the derivation of the values, such failures may be expected by chance alone one time in 50, even if the concrete is controlled exactly as anticipated and is overdesigned to yield an average strength equal to f_{cr}.

Most specifications for concrete strength require that a test be comprised of two or three specimens from the same sample of concrete. The specimens are necessary to obtain a reliable average for a given sample and to provide range data R for determining within-sample variations.

4.4—Quality control charts

Quality control charts have been used by manufacturing industries for many years as an aid in reducing variability and increasing efficiency in production. Methods are well established for the setting up of such charts and are outlined in convenient form in the *ASTM Manual on Quality Control of Materials*.[4] Based on the pattern of previous results and limits established therefrom, trends become apparent as soon as new results are plotted. Points which fall outside the calculated limits indicate that something has affected the control of the process. Such charts are recommended wherever concrete is in continuous production over considerable periods.

Three simplified charts prepared specifically for concrete control are illustrated in Fig. 4.4. While these do not contain all the features of formal control charts they should prove useful to the engineer, architect, and plant superintendent.

(a) A chart in which the results of all strength tests are plotted as received. The line for the required average strength is established as indicated by Eq. (4-1a) or Table 4.3 and the specified design strength.

(b) Moving average for compressive strength where the average is plotted for the previous five sets of two companion cylinders for each day or shift, and the specified strength in this case is the lower limit. This chart is valuable in indicating trends and will show the influence of seasonal changes, changes in materials, etc. The number of tests averaged to plot moving averages with an appropriate lower limit can be varied to suit each job.

(c) Moving average for range where the average range of the previous ten groups of companion cylinders is plotted each day or shift. The maximum average range allowable for good laboratory control is also plotted. Maximum average range is determined as discussed in Section 4.5.

Fig. 4.4 shows Charts (a), (b), and (c) for 46 tests. To be fully effective charts should be maintained throughout the entire job.

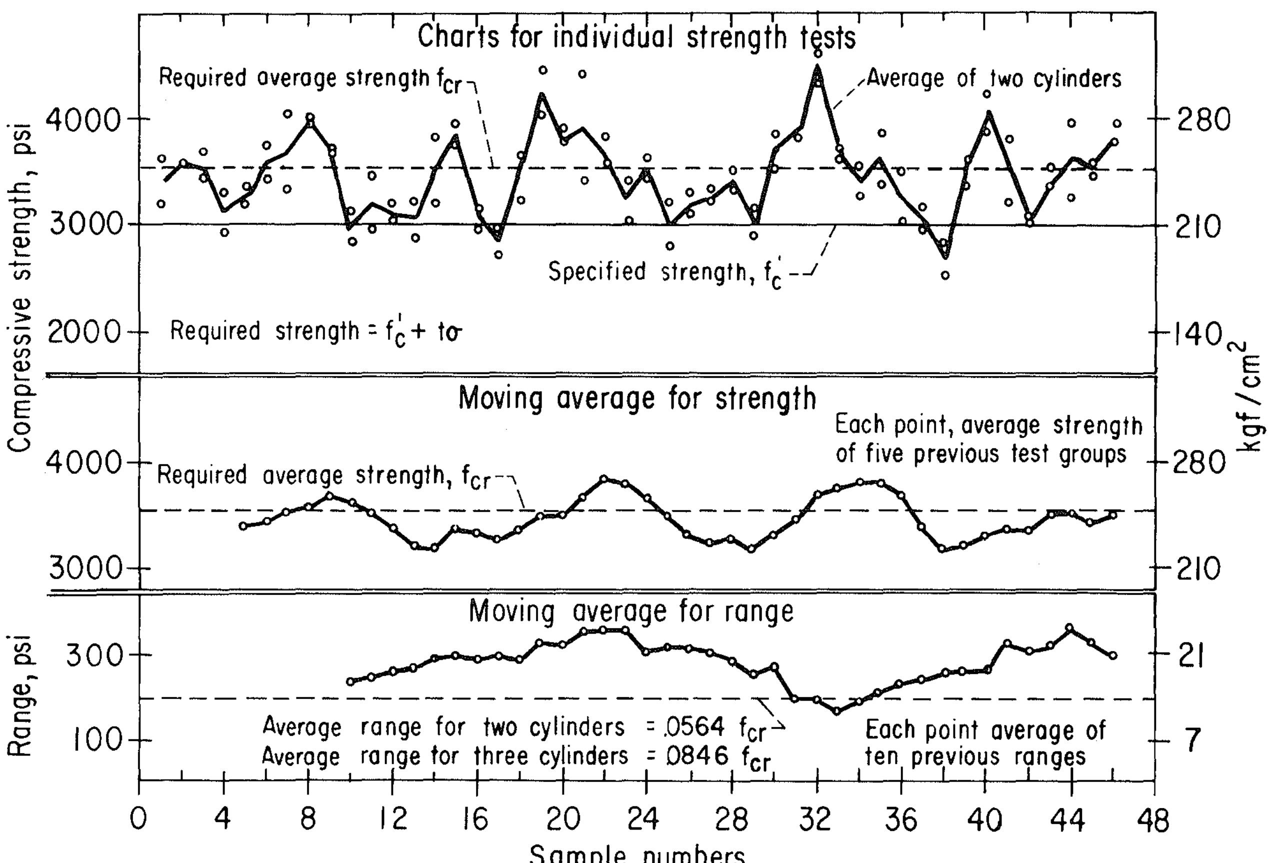

Fig. 4.4—Quality control charts for concrete.

4.5—Tests and specimens required

For any particular job, a sufficient number of tests should be made to insure accurate representation of the variations of the concrete. Concrete tests can be made either on the basis of time elapsed or cubic yardage placed and conditions on each job will determine the most practical method of obtaining the number of tests needed. A test is defined as the average strength of all specimens of the same age fabricated from a sample taken from a single batch of concrete.

A project where all concrete operations are supervised by one engineer provides an excellent opportunity for control and for accurate estimates of reliability with a minimum of tests. Once operations are progressing smoothly tests taken each day or shift, depending on the volume of concrete produced, are sufficient to obtain data which reflect the variations in the concrete of the structure. In general, it is advisable to make a sufficient number of tests so that each different type of concrete placed during any one day will be represented by at least one test which is an average of two standard 6 x 12 in. cylinders tested at the required age. Single specimens taken from two different batches each day will provide more reliable information on overall variations, but it is usually desirable to make companion specimens from the same sample to obtain a check on the within-test variation.

The number of specimens required by the engineer (architect) should be based on established standards but may be reduced as the reliabilities of the producer, the laboratory, and the contractor are established.

The laboratory has the responsibility of making accurate tests, and concrete will be penalized unnecessarily if tests show greater variations or lower average strength levels than actually exist. Since the range between companion specimens from the same sample can be assumed to be the responsibility of the laboratory, a control chart for ranges (Fig. 4.4) should be maintained by the laboratory as a check on the uniformity of its operations. It should be noted that these ranges will not reveal day to day differences in testing, curing, and capping procedures or testing procedures which affect strength levels over long periods. The range between companion cylinders depends on the number of specimens in the group and the within-test variation. This relationship is expressed by the following equation [see Eq. (3-4) and (3-5)]

$$\overline{R}_m = f_{cr} V_1 d_2 \qquad (4\text{-}2)$$

where $\overline{R}_m$ is the average range in Control Chart (c) of Fig. 4.4. The within-test coefficient of variation V_1 should not be greater than 5 percent for good control (Table 3.5), and the estimate of the corresponding average range will be:

$$\overline{R}_m = (0.05 \times 1.128) f_{cr} = 0.05640 f_{cr}$$

for groups of two companion cylinders

$$\overline{R}_m = (0.05 \times 1.693) f_{cr} = 0.08465 f_{cr}$$

for groups of three companion cylinders.

A cylinder of concrete 6 in. in diameter and 12 in. high which has been moist cured for 28 days at 21 C is generally considered a standard specimen for strength and control of concrete if the coarse aggregate does not exceed 2 in. in nominal size. Many times, particularly in the early stages of a job, it becomes necessary to estimate the strength of concrete being produced before the 28-day strength results are available. Concrete cylinders from the same batch should be made and tested at 7 days, or at earlier ages utilizing accelerated test procedures. The 28-day strength can be estimated by extrapolating early test data.

The strength of concrete at later ages, particularly where a pozzolan or cement of slow strength gain is used, is more realistic than the standard 28-day strength. Some structures will not be loaded until concrete has been allowed to mature for longer periods and advantage can be taken of strength gain after 28 days. Some concretes have been found to produce at 28 days less than 50 percent of their ultimate strength. If design is based on strength at later ages, it becomes necessary to correlate these strengths with standard 28-day cylinders since it is not practicable to use later age specimens for concrete acceptance. If possible, the correlation should be established by laboratory tests before construction starts. If mixing plants are located in one place for long enough periods, it is advisable to establish this correlation for reference even though later age concrete is not immediately involved.

Curing concrete test specimens at the construction site and under job conditions is sometimes recommended since this is considered more representative of the curing applied to the structure. These special tests should not be confused with, nor replace, standard control tests. Tests of job-cured specimens may be highly desirable and are necessary when determining the time of form removal, particularly in cold weather, and when establishing the strength of steam-cured concrete pipe, block, and structural members.

The potential strength and variability of concrete can be established by standard 6 x 12 in. cylinders made and cured under standard conditions. Strength specimens of concrete made or cured under other than standard conditions provide additional information but should be analyzed and reported separately.

4.6—Rejection of doubtful specimens

The practice of arbitrary rejection of test cylinders which appear "too far out of line" is not recommended since the normal pattern of probability establishes the possibility of such results. Discarding tests indiscriminately could seriously distort the strength distribution, making analysis of results less reliable.

It occasionally happens that the strength of one cylinder from a group made from a sample deviates so far from the mean as to be highly improbable. It is recommended that a specimen from a test of three or more specimens be discarded if its deviation from a test mean is greater than 3σ, and should be accepted with suspicion if its deviation is greater than 2σ. If questionable variations have been observed during fabrication, curing, or testing of a specimen, the specimen should be rejected. The test average should be computed from the remaining specimens.

A test (average of all specimens of a sample) should never be rejected unless the specimens are known to be faulty, since it represents the best available estimate for the sample.

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Considerations for Design of Concrete Structures Subjected to Fatigue Loading

Reported by ACI Committee 215

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This report presents information that is intended to aid the practicing engineer confronted with consideration of repeated loading on concrete structures. Investigations of the fatigue properties of component materials—concrete, reinforcing bars, welded reinforcing mats, and prestressing tendons—are reviewed. Application of this information to predicting the fatigue life of beams and pavements is discussed.

Keywords: beams (supports); compressive strength; concrete pavements; cracking (fracturing); dynamic loads; **fatigue (materials)**; impact; **loads (forces)**; microcracking; plain concrete; prestressed concrete; prestressing steels; **reinforced concrete**; **reinforcing steels**; **specifications**; static loads; strains; stresses; structural design; tensile strength; welded wire fabric; welding; yield strength.

CONTENTS

*Chairman of ACI Committee 215 at the time that preparation of this report was begun.

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CHAPTER 1—INTRODUCTION

In recent years, considerable interest has developed in the fatigue strength of concrete members. There are several reasons for this interest. First, the widespread adoption of ultimate strength design procedures and the use of higher strength materials require that structural concrete members perform satisfactorily under high stress levels. Hence there is concern about the effects of repeated loads on, for example, crane beams and bridge slabs.

Second, new or different uses are being made of concrete members or systems, such as prestressed concrete railroad ties and continuously reinforced concrete pavements. These uses of concrete demand a high performance product with an assured fatigue strength.

Third, there is new recognition of the effects of repeated loading on a member, even if repeated loading does not cause a fatigue failure. Repeated loading may lead to inclined cracking in prestressed beams at lower than expected loads, or repeated loading may cause cracking in component materials of a member that alters the static load carrying characteristics.

1.1—Objective and scope

This report is intended to provide information that will serve as a guide for design of concrete structures subjected to fatigue loading.

However, this report does not contain the type of detailed design procedures sometimes found in guides.

Chapter 2 presents information on the fatigue strength of concrete and reinforcing materials. This information has been obtained from reviews of experimental investigations reported in technical literature or from unpublished data made available to the committee. The principal aim has been to summarize information on factors influencing fatigue strength that are of concern to practicing engineers.

Chapter 3 considers the application of information on concrete and reinforcing materials to beams and pavements. Provisions suitable for inclusion in a building code are recommended.

An Appendix to this report contains extracts from current specifications that are concerned with fatigue.

1.2—Definitions

It is important to carefully distinguish between static, dynamic, fatigue, and impact loadings. Truly static loading, or sustained loading, remains constant with time. Nevertheless, a load which increases slowly is often called static loading; the maximum load capacity under such conditions is referred to as static strength.

Dynamic loading varies with time in any arbitrary manner. Fatigue and impact loadings are special cases of dynamic loading. A fatigue loading consists of a sequence of load repetitions that may cause a fatigue failure in about 100 or more cycles.

Very high level repeated loadings due to earthquakes or other catastrophic events may cause failures in less than 100 cycles. These failures are sometimes referred to as low-cycle fatigue; however, this report does not specifically deal with these types of loadings.

CHAPTER 2—FATIGUE PROPERTIES OF COMPONENT MATERIALS

The fatigue properties of concrete, reinforcing bars, and prestressing tendons are described in this section. Much of this information is presented in the form of diagrams and algebraic relationships that can be utilized for design. However, it is emphasized that this information is based on the results of tests conducted on different types of specimens subjected to various loading conditions. Therefore, caution should be exercised in applying the information presented in this report.

2.1—Plain concrete*

2.1.1 *General*—Plain concrete, when subjected to repeated loads, may exhibit excessive cracking and may eventually fail after a sufficient number of load repetitions, even if the maximum load is less than the static strength of a similar specimen. The fatigue strength of concrete is defined as a fraction of the static strength that it can support repeatedly for a given number of cycles. Fatigue strength is influenced by range of loading, rate of loading, eccentricity of loading, load history, material properties, and environmental conditions.

Fatigue is a process of progressive permanent internal structural change in a material subjected to repetitive stresses. These changes may be damaging and result in progressive growth of cracks and complete fracture if the stress repetitions are

*Dr. Surendra P. Shah was the chairman of the subcommittee that prepared this section of the report.

sufficiently large.[1,2] Fatigue fracture of concrete is characterized by considerably larger strains and microcracking as compared to fracture of concrete under static loading.[3,4] In this respect, fatigue failure of concrete is not as critical as fatigue failure of metals, which may be of a brittle nature. Fatigue strength of concrete for a life of ten million cycles—for compression, tension, or flexure—is roughly about 55 percent of static strength.

2.1.2 *Range of stress*—The effect of range of stress may be illustrated by the stress-fatigue life curves, commonly referred to as *S-N* curves, shown in Fig. 1. These curves were developed from tests on 6 x 6 in. plain concrete beams[5] loaded at the third points of a 60 in. span. The tests were conducted at the rate of 450 cycles per min. This concrete mix with a water-cement ratio of 0.52 by weight provided an average compressive strength of 5000 psi (352 kgf/cm^2) in 28 days. The age of the specimens at the time of testing ranged from 150 to 300 days.

In Fig. 1, the ordinate is the ratio of the maximum stress, S_{max}, to the static strength. In this case, S_{max} is the computed flexural tensile stress, and the static strength is the modulus of rupture stress, f_r. The abscissa is the number of cycles to failure, plotted on a logarithmic scale.

Curves *a* and *c* indicate that the fatigue strength of concrete decreases with increasing number of cycles. It may be observed that the *S-N* curves for concrete are approximately linear between 10^2 and 10^7 cycles. This indicates that concrete does not exhibit an endurance limit up to 10 million cycles. In other words, there is no limiting value of stress below which the fatigue life will be infinite.

The influence of load range can be seen from comparison of Curves *a* and *c* in Fig. 1. These curves were obtained from tests with loads ranging between a maximum and a minimum which was equal to 75 or 15 percent of the maximum, respectively. It is evident that a decrease of the range between maximum and minimum load results in increased fatigue strength for a given number of cycles. When the minimum and maximum loads are equal, the strength of the specimen corresponds to the static strength of concrete determined under otherwise similar conditions.

The results of fatigue tests usually exhibit substantially larger scatter than static tests. This inherent statistical nature of fatigue test results can best be accounted for by applying probabilistic procedures: for a given maximum load, minimum load, and number of cycles, the probability of failure can be estimated from the test results. By repeating this for several numbers of cycles, a relationship between probability of failure and number of cycles until failure at a given level of

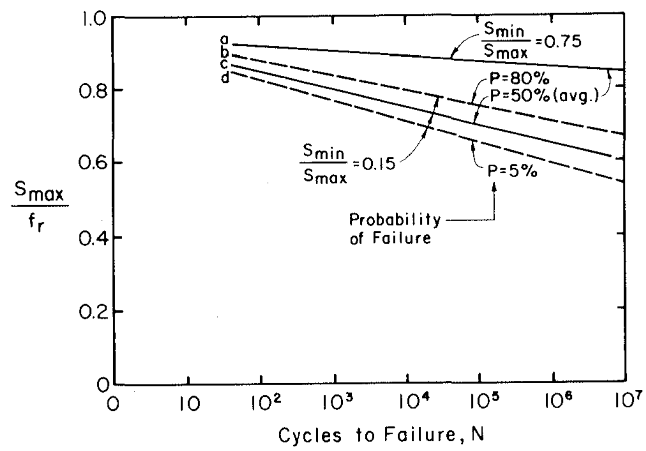


Fig. 1—Fatigue strength of plain concrete beams

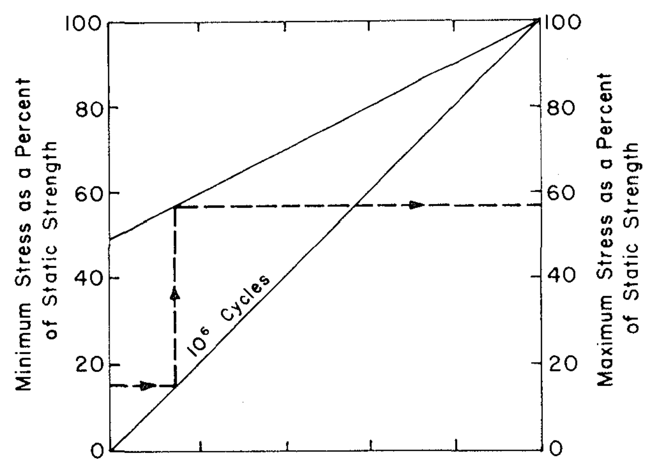


Fig. 2—Fatigue strength of plain concrete in tension, compression, or flexure

maximum load can be obtained. From such relationships, *S-N* curves for various probabilities of failure can be plotted. Curves *a* and *c* in Fig. 1 are averages representing 50 percent probability of failure. Curve *d* represents 5 percent probability of failure, while Curve *b* corresponds to an 80 percent chance of failure.

The usual fatigue curve is that shown for a probability of failure of 50 percent. However, design may be based on a lower probability of failure.

Design for fatigue may be facilitated by use of a modified Goodman diagram, as illustrated in Fig. 2. This diagram is based on the observation that the fatigue strength of plain concrete is essentially the same whether the mode of loading is tension, compression, or flexure. The diagram also incorporates the influence of range of loading. For a zero minimum stress level, the maximum stress level the concrete can support for one million cycles without failure is taken conservatively as 50 percent of the static strength. As the minimum stress level is increased, the stress range that the concrete can support decreases. The linear

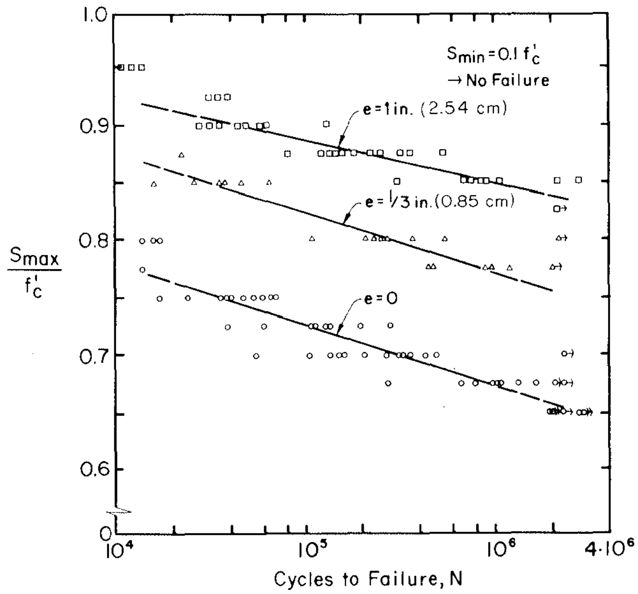


Fig. 3—Influence of stress gradient

decrease of stress range with increasing minimum stress has been observed, at least approximately, by many investigators.

From Fig. 2, the maximum stress in tension, compression, or flexure that concrete can withstand for one million repetitions and for a given minimum stress can be determined. For example, consider a structural element to be designed for one million repetitions. If the minimum stress is 15 percent of the static ultimate strength, then the maximum load that will cause fatigue failure is about 57 percent of static ultimate load.

2.1.3 *Load history*—Most laboratory fatigue data are idealized, since in these tests the loads alternated between constant minimum and maximum values. Concrete in structural members may be subjected to randomly varying loads. Currently, no data are available[6] showing the effect of random loading on fatigue behavior of concrete. Effects of different values of maximum stress can be approximately, although not always conservatively, estimated from constant stress fatigue tests by using the Miner hypothesis.[7] According to this rule, failure occurs if $\Sigma(n_r/N_r) = 1$, where n_r is the number of cycles applied at a particular stress condition, and N_r is the number of cycles which will cause fatigue failure at that same stress condition.

The effect of rest periods and sustained loading on the fatigue behavior of concrete is not sufficiently explored. Laboratory tests have shown that rest periods and sustained loading between repeated load cycles tends to increase the fatigue strength of concrete.[5] In these tests, the specimens were subjected to relatively low levels of sustained stress. If the sustained stress level is above about 75 percent of the static strength, then sustained loading may have detrimental effects on fatigue life.[3] This contradictory effect of creep loading may be explained from test results which show that low levels of sustained stress increase the static strength, whereas high levels of sustained stress resulted in increased microcracking and failure in some cases.

2.1.4 *Rate of loading*—Several investigations indicate that variations of the frequency of loading between 70 and 900 cycles per minute have little effect on fatigue strength provided the maximum stress level is less than about 75 percent of the static strength.[8] For higher stress levels, a significant influence of rate of loading has been observed.[9] Under such conditions, creep effects become more important, leading to a reduction in fatigue strength with decreasing rate of loading.

2.1.5 *Material properties*—The fatigue strength for a life of 10 million cycles of load and a probability of failure of 50 percent, regardless of whether the specimen is loaded in compression, tension, or flexure, is approximately 55 percent of the static ultimate strength. Furthermore, the fatigue strength of mortar and concrete are about the same when expressed as a percentage of their corresponding ultimate static strength.[10] Many variables such as cement content, water-cement ratio, curing conditions, age at loading, amount of entrained air, and type of aggregates that affect static ultimate strength also influence fatigue strength in a similar proportionate manner.[11]

2.1.6 *Stress gradient*—Stress gradient has been shown to influence the fatigue strength of concrete. Results of tests[12] on 4 x 6 x 12 in. concrete prisms under repeated compressive stresses and three different strain gradients are shown in Fig. 3. The prisms had a compressive strength of about 6000 psi (422 kgf/cm^2). They were tested at a rate of 500 cpm at ages varying between 47 and 77 days.

For one case, marked $e = 0$, the load was applied concentrically, producing uniform strain throughout the cross section. To simulate the compression zone of a beam, load was applied eccentrically in the other two cases, marked $e = 1/3$ in. (0.85 cm) and $e = 1$ in. (2.54 cm). The loads were applied such that during the first cycle of fatigue loading the maximum strain at the extreme fiber was the same for all three sets of specimens. For the two eccentrically loaded cases, the minimum strain was zero and half the maximum strain, respectively. The stress level, S, was defined as the ratio of the extreme fiber stress to the static compressive strength f_c'. The extreme fiber stress in eccentrically loaded specimens was determined from static stress strain relationships and the maximum strain at the extreme fiber as observed during the first cycle of fatigue loading.

From the mean *S-N* curves shown in Fig. 3, it can be seen that the fatigue strength of eccentric specimens is 15 to 18 percent higher than that for uniformly stressed specimens for a fatigue life of 40,000 to 1,000,000 cycles. These results are in accord with the results of static tests where it was shown that the strain gradient retards internal microcrack growth.[13] For the purpose of design of flexural members limited by concrete fatigue in compression, it is safe to assume that fatigue strength of concrete with a stress gradient is the same as that of uniformly stressed specimens.

2.1.7 *Mechanism of fatigue fracture*—Considerable research is being done to study the nature of fatigue failure in concrete.[1-4, 14-17] Researchers have measured surface strains, changes in pulse velocity, internal microcracking and surface cracking to understand the phenomenon of fracture. It has been observed that fatigue failure is due to progressive internal microcracking. As a result, large increase in both the longitudinal and transverse strains and decrease in pulse velocity have been reported preceding fatigue failure. External surface cracking has been observed on test specimens long before actual failure.

Progressive damage under fatigue loading is also indicated by reduction of the slope of the compressive stress-strain curve with an increasing number of cycles. In addition to internal microcracking, fatigue loading is also likely to cause changes in the pore structure of the hardened cement paste. Creep effects must also be considered. They become more significant as the rate of loading decreases.

2.1.8 *Concrete strain*—Similar to the behavior of concrete under sustained loads, the strain of concrete during repeated loading increases substantially beyond the value observed after the first load application,[2] as shown in Fig. 4. The strain at fatigue failure is likely to be higher if the maximum stress is lower.

2.2—Reinforcing bars*

2.2.1 *General*—Fatigue of steel reinforcing bars has not been a significant factor in their application as reinforcement in concrete structures. However, the trend in concrete structures toward use of ultimate strength design procedures and higher yield strength reinforcement makes fatigue of reinforcing bars of more concern to designers. It is noteworthy, though, that the lowest stress range known to have caused a fatigue failure of a straight hot-rolled deformed bar embedded in a concrete beam is 21 ksi. This failure occurred after 1,250,000 cycles of loading on a beam containing a #11, Grade 60 test bar, when

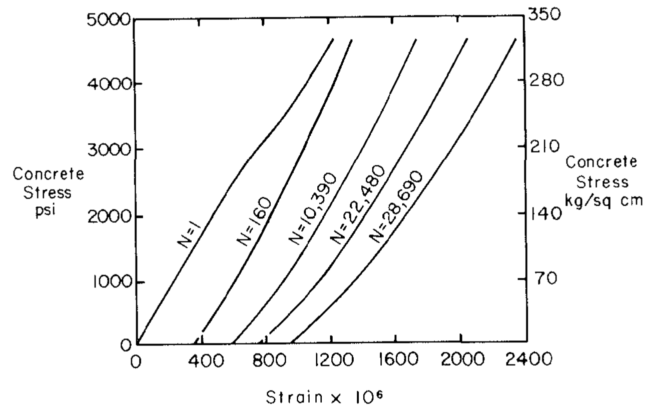


Fig. 4—Effect of repeated load on concrete strain

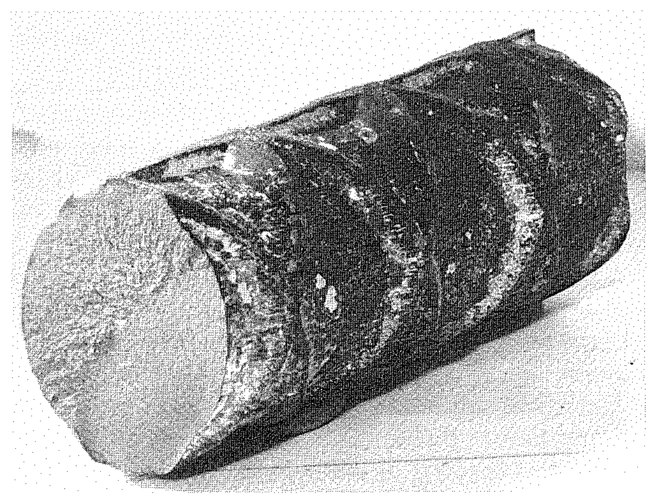


Fig. 5—Fatigue fracture of a reinforcing bar

the minimum stress level was 17.5 ksi (1230 kgf/cm^2).[26]

A typical fatigue fracture of a reinforcing bar is shown in Fig. 5. This is also a #11, Grade 60 bar which at one time was embedded in a concrete beam that was subjected to repeated loads until the bar failed. In this figure, the orientation of the bar is the same as it was in the beam; the bottom of the bar was adjacent to the extreme tensile fibers in the beam. The smoother zone, with the dull, rubbed appearance, is the fatigue crack. The remaining zone of more jagged surface texture is the part that finally fractured in tension after the growing fatigue crack weakened the bar. It is noteworthy that the fatigue crack did not start from the bottom of the bar. Rather it started along the side of the bar, at the base of one of the transverse lugs. This is a common characteristic of most bar fatigue fractures.

Quite a number of laboratory investigations of the fatigue strength of reinforcing bars have been reported in recent years from the United States,[18-26]

*Dr. John M. Hanson was the chairman of the subcommittee that prepared this section of the report.

Canada,[27,28] Europe,[29-34] and Japan.[35-39] In most of these investigations, the relationship between stress range, S_r, and fatigue life, N, was determined by a series of repeated load tests on bars which were either embedded in concrete or tested in air.

There is contradiction in the technical literature as to whether a bar has the same fatigue strength when tested in air or embedded in a concrete beam. In an investigation[31] of hot-rolled cold-twisted bars, it was found that bars embedded in beams had a greater fatigue strength than when tested in air. However, in another investigation,[29] the opposite conclusion was reached. More recent studies[28,32] indicate that there should be little difference in the fatigue strength of bars in air and embedded bars if the height and shape of the transverse lugs are adequate to provide good bond between the steel and concrete.

The influence of friction between a reinforcing bar and concrete in the vicinity of a crack has also been considered.[32] In laboratory tests, an increase in temperature is frequently observed at the location where the fatigue failure occurs. However, rates of loading up to several thousand cycles per minute and temperatures up to several hundred degrees C are normally not considered to have a significant effect on fatigue strength.[40] In a statistical analysis[41] of an investigation of reinforcing bars,[26] differences in fatigue strength due to rates of loading of 250 and 500 cycles per minute were not significant.

It is therefore believed that most of the data reported in investigations in North America and abroad is directly comparable, even though it may have been obtained under quite different testing conditions.

A number of S_r-N curves obtained from tests on concrete beams containing straight deformed bars made in North America[18,21,24-28] are shown in Fig. 6. These curves are for bars varying in size from #5 to #11, with minimum stress levels ranging from —0.10 to 0.43 of the tensile yield strength of the bars.

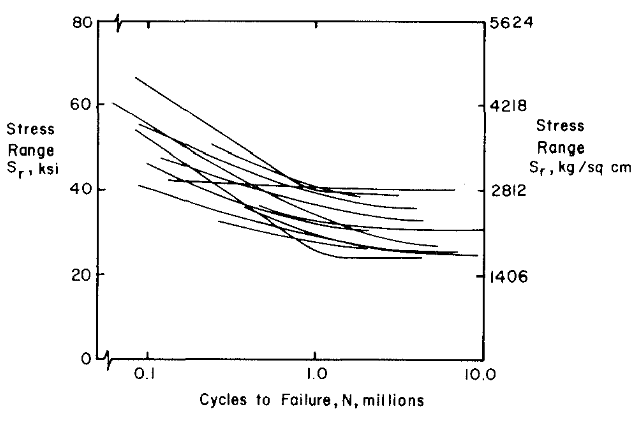


Fig 6—Stress range—fatigue life curves for reinforcing bars

Although only about one-third of the total number of S_r-N curves reported in the indicated references are shown in Fig. 6, they include the highest and lowest fatigue strength. The varying characteristics of these curves suggest that there are many variables in addition to stress range that influence the fatigue strength of deformed reinforcing bars.

Most of the curves in Fig. 6 show a transition from a steeper to a flatter slope in the vicinity of one million cycles, indicating that reinforcing bars exhibit a practical fatigue limit. Fatigue strengths associated with the steeper or flatter part of the S_r-N curves will be referred to as being in the finite life or long life region, respectively. Because of the lack of sufficient data in the long life region, it is noted that many of the S_r-N curves in this region are conjectural.

The fatigue strength of the steel in reinforcing bars depends upon chemical composition, microstructure, inclusions, and other variables.[40] However, it has been shown[26,28] that the fatigue strength of reinforcing bars may be only one-half of the fatigue strength of coupons machined from samples of the bars. In addition, reinforcing bar specifications are based on physical characteristics. Consequently, the variables related to the steel composition are of limited concern to practicing structural engineers. The variables related to the physical characteristics and use of the reinforcing bars are of greater concern. The main variables that have been considered in the technical literature are:

1. Minimum stress
2. Bar size and type of beam
3. Geometry of deformations
4. Yield and tensile strength
5. Bending
6. Welding

Each of these is discussed in the following sections.

2.2.2 *Minimum stress*—In several investigations,[18,21,29] it has been reported that the fatigue strength of reinforcing bars is relatively insensitive to the minimum stress level. However, in two recent investigations,[26,28] it was concluded that minimum stress level does influence fatigue strength to the extent approximately indicated by a modified Goodman diagram with a straight line envelope. This indicates that fatigue strength decreases with increasing minimum stress level in proportion to the ratio of the change in the minimum stress level to the tensile strength of the reinforcing bars.

2.2.3 *Bar size and type of beam*—These two factors are related because bars embedded in concrete beams have a stress gradient across the bar. In design, it is only the stress at the midfibers of the bar that is generally considered. Large bars in shallow beams or slabs may have a significantly higher stress at the extreme rather than the midfibers of the bar.

The effect of bar size is examined in Table 1 using data from three investigations.[28,32,36] Since #8 bars or their equivalent were tested in each of these investigations, the fatigue strength of other bar sizes was expressed as a ratio relative to the fatigue strength of the #8 bars. For each comparison, the bars were made by the same manufacturer, and they also were tested at the same minimum stress level. The fatigue strength is the stress range causing failure at 2 million or more cycles.

TABLE 1—EFFECT OF BAR SIZE

Tests reported in	Grade of bar	Fatigue strength relative to fatigue strength of No. 8 bars			
		No. 5	No. 6	No. 8	No. 10
Reference 28	40	1.06	—	1.00	0.99
	60	1.08	—	1.00	0.96
	75	1.20	—	1.00	0.95
Reference 32	40	1.11	—	1.00	—
	40	1.05	—	1.00	—
	60	1.05	—	1.00	—
	75	1.10	—	1.00	—
Reference 36	40	—	1.12	1.00	—
	60	—	1.04	1.00	—
	60	—	1.10	1.00	—

The tests reported in Reference 32 were on bars subjected to axial tension. Therefore, there was no effect of strain gradient in this data, yet the fatigue strength of the #5 bars was about 8 percent greater than that of the #8 bars.

Tests in Reference 28 were on bars in concrete beams. The strain gradients in these beams resulted in stresses at the extreme fibers for the different size bars that were about the same. Still, an effect of bar size was found that was of about the same order of magnitude.

In the tests in Reference 36 the strain gradient was greater across the #8 bars than the #6 bars. Therefore, part of the difference in fatigue strength should be attributed to the higher stress at the extreme fibers of the #8 bars. However, the differences, compared to the other test results, are about the same.

In another investigation[26,41] where both bar size and type of beam were controlled variables, the former was found to be significant and the latter was not significant. This investigation included bars of 5 different sizes—#5, 6, 8, 10, and 11—made by a major United States manufacturer. These bars were embedded in rectangular or T-shaped concrete beams having effective depths of 6, 10, or 18 in. (15.2, 25.4, or 45.7 cm). In this investigation, the fatigue life of #8, Grade 60 bars subjected to a stress range of 36 ksi (2530 kgf/cm^2) imposed on a minimum stress of 6 ksi (422 kgf/cm^2) was 400,000 cycles. Under identical stress conditions, the fatigue life of the #5, 6, 10, and 11 bars were found to be 1.22, 1.30, 0.76, and 0.85 times the life of the #8 bars, respectively. This trend is the same as that for the data shown in Table 1. The irregular variation was attributed to differences in surface geometry.

2.2.4 *Geometry of deformations*—Deformations on reinforcing bars provide the means of obtaining good bond between the steel and the concrete. However, these same deformations produce stress concentrations at their base, or at points where a deformation[20,21,23] intersects another deformation or a longitudinal rib. These points of stress concentrations are where the fatigue fractures are observed to initiate.

Any evaluation of the influence of the shape of the deformations on fatigue properties of the bar must recognize that the rolling technique and the cutting of the rolls necessarily requires specific limitations and variations in the pattern. This applies to the height of the deformations, the slopes on the walls of the deformations, and also to the fillets at the base of the deformations.

An analytical study[42] has shown that stress concentration of an external notch on an axially loaded bar may be appreciable. This study indicated that the width, height, angle of rise, and base radius of a protruding deformation affect the magnitude of the stress concentration. It would appear that many reinforcing bar lugs may have stress concentration factors of 1.5 to 2.0.

Tests on bars having a base radius varying from about 0.1 to 10 times the height of the deformation have been reported.[25,26,28,36] These tests indicate that when the base radius is increased from 0.1 to about 1 to 2 times the height of the deformation, fatigue strength is increased appreciably. An increase in base radius beyond 1 to 2 times the height of the deformation does not show much effect on fatigue strength. However, Japanese tests[36] have shown that lugs with radii larger than 2 to 5 times the height of the deformation have reduced bond capacity.

Tests have indicated[30,31,39] that decreasing the angle of inclination of the sides of the deformations with respect to the longitudinal axis increases the fatigue strength of a reinforcing bar. This increase occurs for bars with lugs having abrupt changes in slope at their bases. It has been

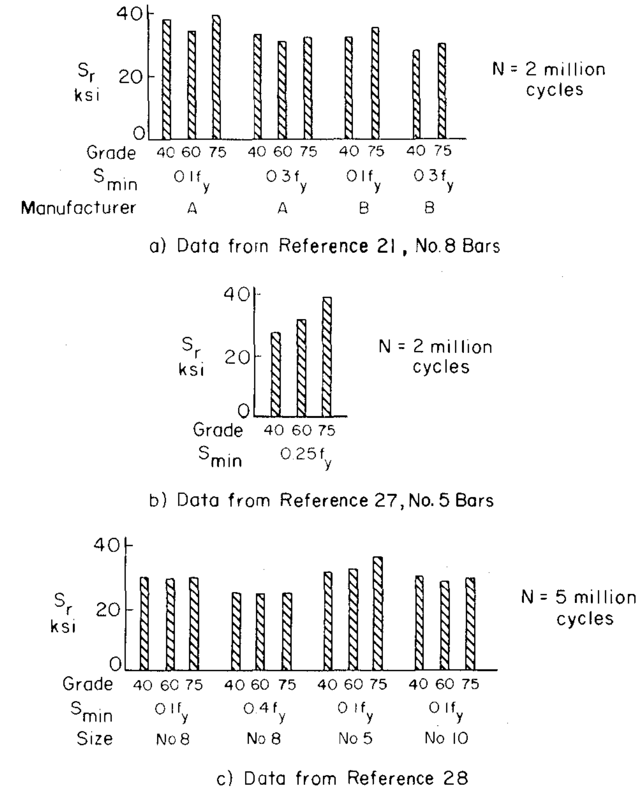


Fig. 7—Effect of grade of bar

noted[13] that the base radius should be determined in a plane through the longitudinal axis of the bar, since this is the direction of the applied stress. The base radius determined in this plane will be substantially larger than a base radius determined in a plane perpendicular to a sharply inclined lug.

In two experimental investigations,[23,34] it was found that the condition of the rolls, whether new or worn, had little effect on fatigue strength. However, a conflicting opinion has been expressed in Reference 32.

Tests[32] also show a substantial effect on the fatigue resistance of reinforcing bars due to brand marks. The brand marks cover the identification of the bar as to size, type of steel (billet, rail, or axle), mill that rolled the steel, and yield strength (Grade 40, 60, or 75).[44] The stress concentration at a bar mark is similar to that caused by bar deformations.

It has also been demonstrated[24] that the fatigue strength of a reinforcing bar may be influenced by the orientation of the longitudinal ribs. In that study, an increased fatigue life was obtained when the longitudinal ribs were oriented in a horizontal position rather than a vertical position. This phenomenon is apparently associated with the location at which the fatigue crack initiates. In other words, if there is a particular location on the surface of a bar which is more critical for fatigue than other locations, then the positioning of that location in the beam will influence the fatigue strength.

2.2.5 *Yield and tensile strength*—In three investigations,[21,27,28] the fatigue strength of different grades[44] of bars made by the same North American manufacturer were compared. The results of these comparisons, all of which are in the long life region of fatigue life, are shown by the bar graphs in Fig. 7. It was concluded in References 21 and 28 that the fatigue strength of the bars was relatively insensitive to their yield or tensile strength. References 21 and 28 include 157 and 72 tests, respectively. Reference 27, which includes 19 tests, indicated that fatigue strength may be predicted for grade of steel as a function of the stress range.

In another investigation[26,41] on bars made by a major United States manufacturer, the fatigue life of Grade 40, Grade 60, and Grade 75 #8 bars, subjected to a stress range of 36 ksi (2530 kgf/cm^2) imposed on a minimum stress of 6 ksi (422 kgf/cm^2), varied linearly in the ratio of 0.69 to 1.00 to 1.31, respectively. The ratio of 1.0 corresponds to a fatigue life of 400,000 cycles, and is therefore in the finite life region.

Axial tension fatigue tests[32] on unembedded reinforcing bars made in Germany were carried out on four groups of bars having yield strengths of 49, 53, 64, and 88 ksi (3445, 3726, 4499, and 6186 kgf/cm^2). All of the bars were rolled through the same stand for elimination of variation in the deformed surfaces. When tested with a minimum stress level of 8.5 ksi (598 kgf/cm^2), the stress ranges causing failure in two million cycles were determined to be 28, 28, 28, and 31 ksi (1968, 1968, 1968, and 2179 kgf/cm^2), respectively.

In a Japanese investigation,[36] bars of the same size and made by the same manufacturer but with yield strengths of 50, 57, and 70 ksi (3515, 4007, and 4921 kgf/cm^2) were tested. The stress range causing failure in two million cycles was between 30 and 31.5 ksi (2109 and 2214 kgf/cm^2) for all three groups of bars.

2.2.6 *Bending*—The effect of bends on fatigue strength of bars has been considered in two investigations.[21,29] In the North American investigation,[21] fatigue tests were carried out on both straight and bent #8 deformed bars embedded in concrete beams. The bends were through an angle of 45 deg around a pin of 6 in. (15.2 cm) diameter. The fatigue strength of the bent bars was a little more than 50 percent below the fatigue strength of the straight bars. In one test, a bent bar embedded in a reinforced concrete beam failed in fatigue after sustaining 900,000 cycles of a stress range of 18 ksi (1265 kgf/cm^2) imposed on a minimum stress of 5.9 ksi (415 kgf/cm^2). In another test, application of 1,025,000 cycles produced

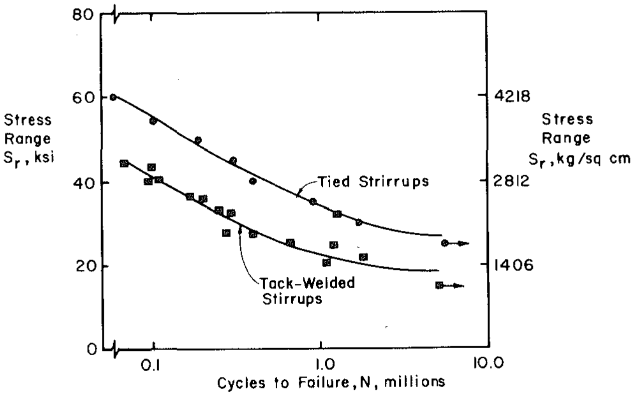


Fig. 8—Effect of tack welding stirrups to Grade 60 bars

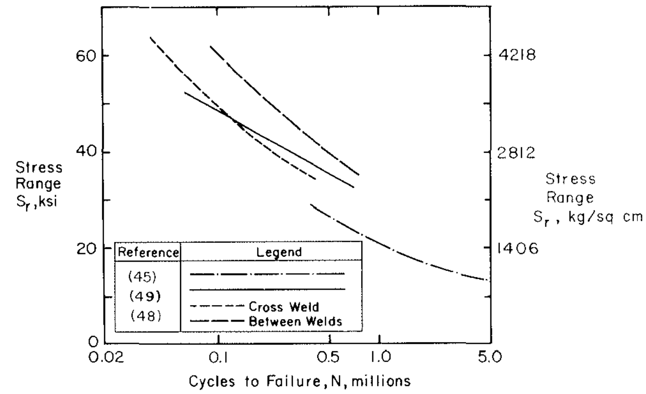


Fig. 9—Median S_r-N curves for welded reinforcing mats

a failure when the stress range and minimum stress were 16.4 ksi and 19.1 ksi (1153 and 1343 kgf/cm^2), respectively.

Tests[29] have also been reported from Germany on both plain and deformed hot-rolled bars bent through an angle of 45 deg. However, these bars were bent around a pin having a diameter of 10 in. (25.4 cm). Compared to tests on straight bars, the fatigue strength of the plain bars was reduced 29 percent by the bend, while the fatigue strength of the deformed bars was reduced 48 percent.

2.2.7 *Welding*—In an investigation[24] using Grade 40 and Grade 60 reinforcement with the same deformation pattern, it was found that the fatigue strength of bars with stirrups attached by tack welding was about one-third less than bars with stirrups attached by wire ties. The results of the tests on the Grade 60 reinforcement are shown in Fig. 8. For both grades of steel, the fatigue strength of the bars with tack welding was about 20 ksi (1406 kgf/cm^2) at 5 million cycles. All of the fatigue cracks were initiated at the weld locations.

Investigations[19,22] have also been carried out to evaluate the behavior of butt-welded reinforcing bars in reinforced concrete beams. In tests conducted at a minimum stress level of 2 ksi tension, the least stress range that produced a fatigue failure was 24 ksi. It was observed that minimum stress level in the butt-welded joint was not a significant factor affecting the fatigue strength of the beams.

2.3—Welded wire fabric and bar mats*

Welded wire fabric may consist of smooth or deformed wires while bar mats usually consist of deformed bars. Often fabric and bar mats are not used in structures subject to significant repeated loads because of concern that the welded intersections will create significant stress concentrations. This feeling has been heightened by experience from abroad[45] and the relatively poor performance of smooth wire fabric in continuously reinforced concrete pavements.[46,47,48] In some cases, pavements reinforced with this fabric performed adequately in service for 3 to 5 years. Then several wide cracks occurred, necessitating extensive repairs. While most of this cracking was caused by inadequate detailing of splices, field studies in Connecticut[47] have revealed failures at the welds in a significant number of instances.

Any assessment of welded wire fabric or bar mats based primarily on their performance in pavements is unrealistic. In any given length of pavement, wide variations are possible in the stress spectrum for the reinforcement. The average stress level in the reinforcement is strongly dependent on the pavement's age, its thermal and moisture history, and the longitudinal restraint offered by the subgrade. The stress range in the reinforcement caused by the traffic depends on the support offered by the subgrade as well as the magnitude of the loading.

Several recent investigations have examined the fatigue characteristics of fabric and bar mats in air.[45,48,49] For smooth wire fabric[45,49] the disturbance due to the welded intersection dominated over all other influences, so that failures were confined to the heat affected zone of the weld. For bar mats, the disturbance due to the welded intersection dominated only if the stress concentration caused by the intersection was greater than the concentration caused by the deformation. The available evidence does not indicate that these effects are additive.

Results for "cross-weld" tests conducted in air are summarized in Fig. 9. In the German investigation[45] 15 tests were made on a smooth wire fabric consisting of 0.236 in. (6 mm) diameter wires welded to 0.315 in. (8 mm) diameter wires.

*Dr. Neil M. Hawkins prepared this section of the report.

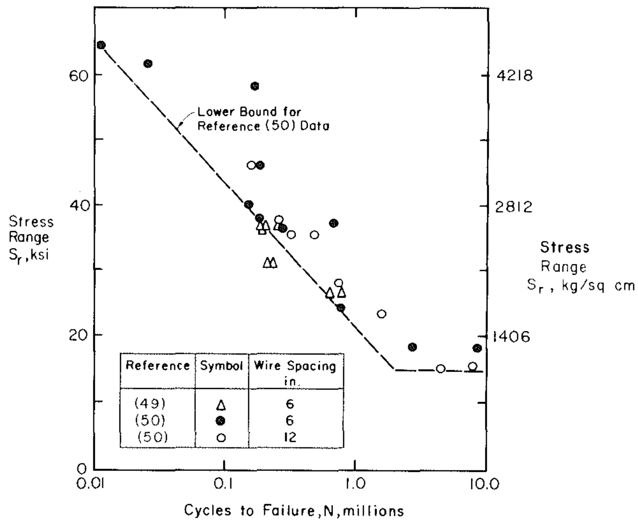


Fig. 10—S_r-N curves for slabs containing mats

In one American investigation[49] 59 "cross-weld" tests were made on a 2 x 2-6 x 6 (0.263 in. or 6.7 mm diameter) smooth wire fabric, and in the other investigation[48] 22 "cross-weld" tests and 30 between weld tests were made on #5 Grade 60 deformed bars with #3 deformed bars welded to them.

The University of Washington[49] investigation was intended to provide a statistically analyzable set of test data for three stress ranges. It was observed that when the penetration across the weld was less than one-tenth of the diameter of the wire, there was incomplete fusion of the wires and the formation of a cold joint. For a greater penetration, the molten metal squirted into the intersection between the wires causing a marked stress concentration so that the fatigue life for a hot joint was about half that for a cold joint. The result shown in Fig. 9 is the median fatigue life value for the penetration considered as a random variable. In those tests the fatigue life values for a given stress range and a 95 percent probability of survival exceeded the life values obtained in tests on high yield deformed bars.[25] In the tests[48] on the bar mats it was found that the welded intersection reduced the fatigue life for a given range by about 50 percent throughout the short life stress range.

Tests on slabs reinforced with smooth wire mats have been reported in References 49 and 50. The results are summarized in Fig. 10, where it is apparent that there is reasonable correlation between the two sets of data. In the Illinois tests,[50] the 12 in. (30.5 cm) wide, 60 in. (152 cm) long slabs were reinforced with #0 gage wires longitudinally with #8 gage wires welded to them at 6 or 12 in. (15.3 or 20.6 cm) spacings.

In the University of Washington tests,[49] the 54 in. (137 cm) square slabs were reinforced with two layers of the same 2 x 2-6 x 6 fabric as that tested in air. In the slab tests, it was observed that there was a rapid deterioration of the bond between the smooth wires and the concrete under cyclic loading, so that after 10^4 cycles of loading, all anchorage was provided primarily by the cross wires. Fatigue life values for fracture of the first wire in those slabs could be predicted using the results for the wire tested in air and a deterministic assessment of the appropriate probability based on the number of approximately equally stressed welds in the slab. The appropriate probability level for these slabs was about 98 percent, indicating a need for a design approach for welded reinforcing mats based on a probability of survival greater than the 95 percent commonly accepted for reinforcing bars and concrete.

The fatigue life values for collapse were about double those for fracture of the first wire. The values for collapse could be predicted from the results of the tests conducted in air using a deterministic procedure for assessment of the appropriate probability level and Miner's theory[7] to predict cumulative damage effects.

A comparison of the S-N curves for wire fabric and bar mats with those for deformed bars indicates that an endurance limit may not be reached for the fabric and mats until about 5×10^6 cycles, whereas a limit is reached for the bars at about 1×10^6 cycles. However, the total amount of data in the long life range for fabric and mats is extremely limited and insufficient for reliable comparison.

2.4—Prestressing tendons*

2.4.1 *General*—If the precompression in a prestressed concrete member is sufficient to ensure an uncracked section throughout the service life of the member, the fatigue characteristics of the prestressing steel and anchorages are not likely to be critical design factors. Further, in a properly designed unbonded member, it is almost impossible to achieve a condition for which fatigue characteristics are important.[51] Consequently, fatigue considerations have not been a major factor in either the specification of steel for prestressed concrete[52] or the development of anchorage systems.

No structural problems attributable to fatigue failures of the prestressing steel or anchorages have been reported in North America. However, in the near future fatigue considerations may merit closer scrutiny due to:

*Dr. Neil M. Hawkins was chairman of the subcommittee that prepared this section of the report.

1. The acceptance of designs[53] which can result in a concrete section cracked in tension under loads, and

2. the increasing use of prestressing in marine environments, railroad bridges, machinery components, nuclear reactor vessels, railroad crossties, and other structures subject to frequent repeated loads which may involve high impact loadings or significant overloads.

In the United States, the growing concern with the fatigue characteristics of the prestressing system is reflected in several design recommendations developed recently. As a minimal requirement appropriate for unbonded construction, ACI-ASCE Committee 423,[54] ACI Committee 301,[55] and the PCI Post-Tensioning Committee[56] have recommended that tendon assemblies consisting of prestressing steel and anchorages be able to withstand, without failure, 500,000 cycles of stressing varying from 60 to 66 percent of the specified ultimate strength of the assembly. Abroad, standards specifying fatigue characteristics for the tendons have been published in Germany[57] and Japan.[58]

This report does not consider conditions where unbonded prestressing steels and their anchorages are subjected to high impact, low cycle, repeated loadings during an earthquake. ACI-ASCE Committee 423[54] and the PCI Post-Tensioning Committee[56] have developed design recommendations for that situation.

Many factors can influence the strength measured in a fatigue test on a tendon assembly. The tendon should be tested in the "as delivered" condition and the ambient temperature for a test series maintained within ± 3 F (± 1.7 C). The length between anchorages should be not less than 100 times the diameter of the prestressing steel, eight times the strand pitch or 40 in. Test conditions must not cause heating of the specimen, especially at the anchorages, so that a frequency of 200 to 600 cpm is desirable.[59]

Many variables affect the fatigue characteristics of the prestressing system. Within commercially available limits, the designer can specify the following:

1. Type of prestressing steel (wire, strand, or bar)
2. Steel treatment
3. Anchorage type
4. Degree of bond

Seven-wire strand was developed in the United States, while most other prestressing systems are of European origin. Therefore, in the United States, attention has been focused mainly on the fatigue characteristics of seven-wire strand. Recent data on the fatigue characteristics of foreign systems has been summarized by Baus and Brenneisen.[59]

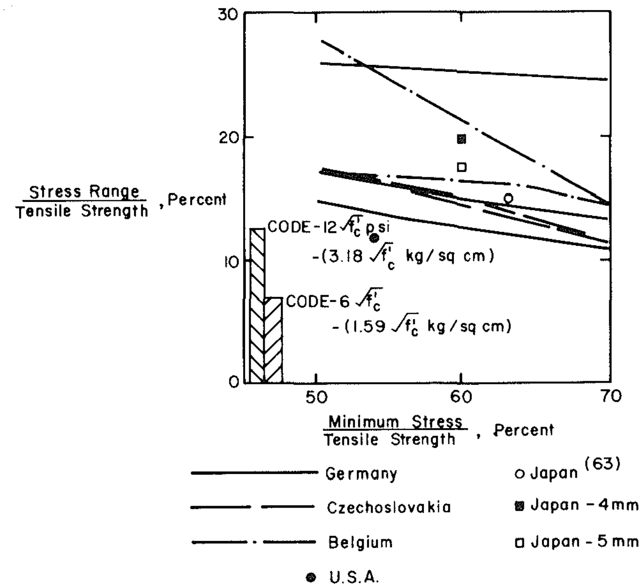


Fig. 11—Fatigue strength at two million cycles for wires

2.4.2 *Type of prestressing steel*—Prestressing steels can be classified into three basic types: wire, strand, and bars. Wires are usually drawn steels and strands are manufactured from wires. Bars are usually hot-rolled alloy steels. Wires are usually made from a steel whose principal alloying components are about 0.8 percent carbon, 0.7 percent manganese, and 0.25 percent silicon. Hot-rolled alloy steels contain about 0.6 percent carbon, 1.0 percent manganese and 1.0 percent chromium. Typically, hot-rolled steels have a tensile strength of 160 ksi (11,250 kgf/cm^2) while drawn wires have strengths ranging between about 250 and 280 ksi (17,580 and 19,680 kgf/cm^2). Drawing increases the tensile strength of the wire. It produces a grain structure which inhibits crack nucleation and provides a smooth surface which reduces stress concentrations. Consequently, the fatigue strengths of wires for a given number of cycles are higher than those of rolled steels. However, the differences are small for stress ranges expressed as percentages of the ultimate tensile strengths.

Wires—Wires of United States manufacture conform to ASTM Designation: A 421-65,[60] "Specifications for Uncoated Stress Relieved Wire for Prestressed Concrete." This specification covers plain wires only. Ribbed varieties are in common use abroad. The fatigue characteristics of wires vary greatly with the manufacturing process, the tensile strength of the wire, and the type of rib. In Fig. 11, fatigue strengths are shown for 2×10^6 cycles for tests performed in Germany, Czechoslovakia, and Belgium,[59] and Japan.* The solid

*Personal communication from Dr. A. Doi, Shinko Wire Co., Ltd., Amagasaki, Hyogo, Japan.

circle in Fig. 11 is the result of a limited series of tests on 0.25 in. (6.3 mm) diameter wires of United States manufacture.[61] These tests showed a fatigue strength at 4×10^6 cycles in excess of 30 ksi (2100 kgf/cm^2). The squares are results for tests on 4 and 5 mm diameter wires performed by the Shinko Wire Company.

Also shown in Fig. 11 are likely ranges in stress for bonded beams designed in accordance with the ACI Code. The lower value is about the maximum possible when the tensile stress in the precompressed zone is limited to $6\sqrt{f_c'}$ psi ($1.6\sqrt{f_c'}$ kgf/cm^2), so that the section is uncracked. The upper value is about the maximum possible when the tensile stress is limited to $12\sqrt{f_c'}$ psi ($3.18\sqrt{f_c'}$ kgf/cm^2) so that the section may contain a crack as wide as 0.005 in. (0.125 mm). It can be seen that although the characteristics of wires vary widely, all could probably be justified for use with a limiting stress of $12\sqrt{f_c'}$ psi.

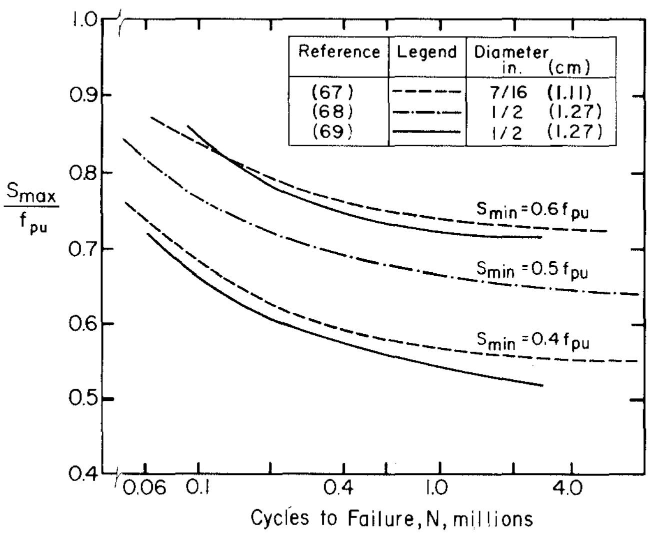


Fig. 12—Data for United States made seven-wire strand

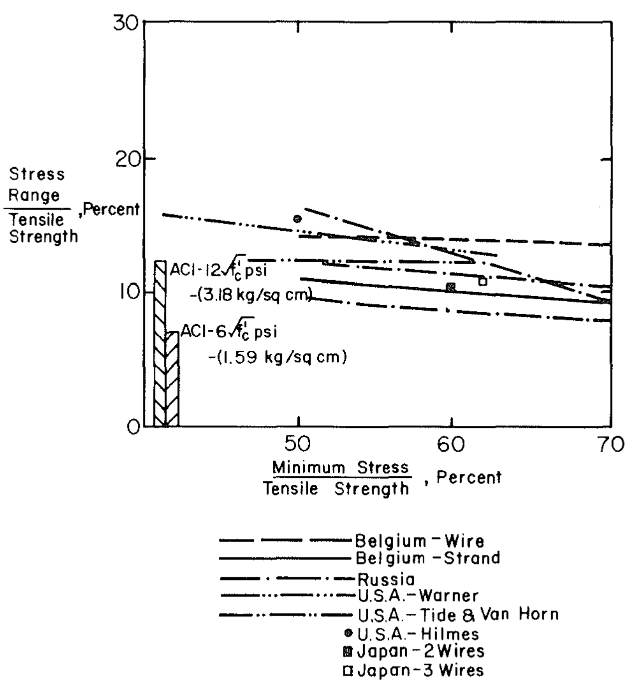


Fig. 13—Fatigue strength at two million cycles for prestressing strand

In Czechoslovakia, tests on plain wires of 3, 4.5, and 7 mm (0.076, 0.114, and 0.127 in.) diameter have shown that within 5 percent, the fatigue characteristics of these wires were independent of the wire diameter.

The effects of ribbing and indentations on fatigue characteristics have been studied in Great Britain,[62] Germany,[59] Russia,[59] and Japan.[63] These tests have shown that the characteristics depend on the height of the rib, its slope and, most of all, the sharpness of the radii at the base of the rib. With a 0.3 mm (0.012 in.) rib height, a 45 deg slope, and no radius at the base of the rib, the theoretical stress concentration factor was 2.0, and there was a 57 percent reduction in the fatigue strength.[59] This reduction decreased with a decreasing stress concentration factor until for the same rib height obtained using a circular cut out of 10 mm (0.4 in.) radius, the stress concentration factor was 1.36, and there was no reduction in the fatigue strength. Wires crimped[62] with a pitch of 2 in. (5.1 cm) and a crimp height of at least 15 percent of the wire diameter in the unstressed condition, showed a fatigue strength 20 percent lower than that of the plain wire.

Strand—Strands of United States manufacture up through ½ in. (12.7 mm) diameter conform to ASTM A 416-68[64] "Specifications for Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete." This specification covers strand used for prestressing in the United States, and foreign suppliers conform to these requirements. In the United States, several series of tests[65-69] have been made on seven-wire strand of either 7/16 or ½ in. (1.1 or 1.27 cm) diameter. Fatigue data compiled from these studies[68] are shown in Fig. 12. These data are shown along with data obtained from tests on Russian,[59] Belgian,[59] and Japanese[63] strand, in Fig. 13.

The Japanese tests[63] indicated by squares were conducted on 3 mm (0.118 in.) diameter plain wires. Tests on similar size strand made from deformed wires showed strengths about 15 percent lower. Comparison of Fig. 11 and 12 and the results of the Belgian tests indicate the stress ranges available with strand are less than those for wire. The United States and Russian tests indicate a decrease in fatigue strength with increasing size for the wires in the strand. Several writers[59] have hypothesized that for strands the successive lengthening and shortening of the cables produces alternating tensions in the individual wires. Failures initiate where the neighboring wires rub together under this alternating load.

Bars—Bars of United States manufacture conform to the requirements of the PCI Post-Tensioning Committee.[56] Although fatigue tests on such bars have been made* most published information is for European bars less than 0.7 in. (1.8 cm) in diameter. Bars manufactured in the United States range between ¾ and 1⅜ in. (1.9 and 3.5 cm) in diameter. Tests on bars ranging between 1 and 1⅜ in. (2.5 and 3.5 cm) in diameter have shown that the fatigue limits of these bars are in excess of 0.1 times the tensile strength of the bar for 1×10^6 cycles of loading at a minimum stress of 0.6 times the tensile strength. As with other post-tensioning systems, the characteristics of the anchorage and not the prestressing system control the fatigue characteristics of the unbonded tendon.

German and Russian tests[59] have shown that the fatigue characteristics for their bars, expressed as a percentage of their ultimate tensile strength, are similar to those of their strand. Tests in Russia on bars with tensile strengths of about 150 ksi (10,540 kgf/cm^2) have shown the fatigue characteristics to be independent of bar size for bar diameters ranging between 0.4 and 0.7 in. (1.0 and 1.8 cm). In Great Britain tests[70] have been made on bonded and unbonded beams post-tensioned with ½ in. (1.25 cm) diameter bars anchored by nuts on tapered threads. There were no fatigue failures of either the bar or the anchorage for 2×10^6 cycles of a loading for which the stress range in the bonded bar was about 12 ksi (844 kgf/cm^2) at a minimum stress equal to at least 60 percent of the bar's static strength.

2.4.3 *Statistical considerations*—Reliable design information requires the collection of the test data in such a manner that statistical methods can be used to define the properties of the material and to investigate the effects of differing parameters.[71,72] At least six and preferably 12 tests are necessary at each stress level to establish fatigue strengths for survivals ranging from 90 to 10 percent. To establish the finite-life part of the *S-N* diagram for a constant minimum stress, tests should be made at a minimum of three stress levels, one near the static strength, one near the fatigue limit, and one in between. Special techniques are needed to establish the fatigue limit.

The overall scatter of fatigue data is of paramount importance in defining the quality of the prestressing steel. For United States strand, a modified Goodman diagram has been developed by Hilmes and Ekberg[68] for three discrete probability levels. As shown in Fig. 14, these levels correspond to survival probabilities of 0.1, 0.5,

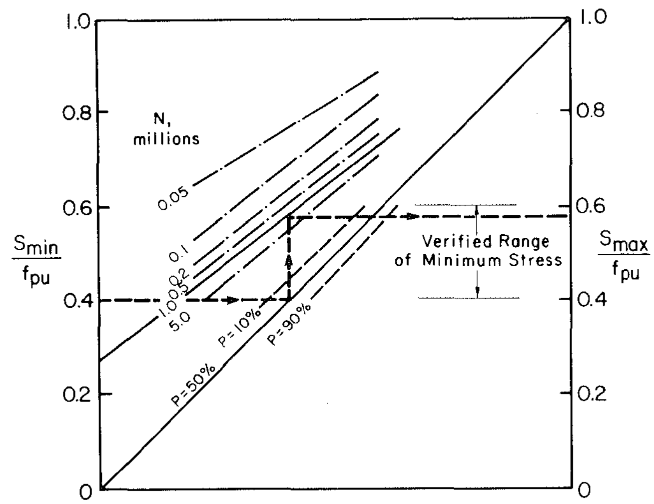


Fig. 14—Strength envelopes for strand tested in United States

and 0.9, and they were developed from data with minimum stress levels of 0.4, 0.5, and 0.6 times the static tensile strength. For the desired minimum stress and probability level, vertical intercepts within Fig. 14 define permissible stress ranges for failure for strands tested in the United States at 5×10^6, 1×10^6, 5×10^5, 2×10^5, 1×10^5, and 5×10^4 cycles.

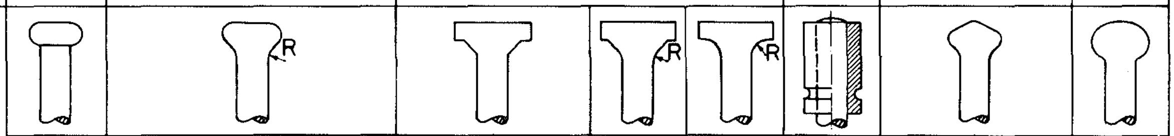
2.4.4 *Steel treatment*—While all United States prestressing steels are stress-relieved, some of those manufactured abroad are not. Czechoslovakian and Russian tests[59] have shown that stress-relieving increases the fatigue limit significantly.

For applications external to a member, the prestressing steel is sometimes protected by hot dip galvanizing. Galvanizing can result in hydrogen embrittlement[73] and therefore its use in structures where fatigue is a consideration is not recommended. For wires and strand, galvanizing reduces the ultimate and yield strength significantly[73] and therefore also reduces the fatigue limit. For bars, galvanizing does not alter the static properties, but it does reduce the fatigue limit.

2.4.5 *Anchorage type*—For unbonded construction, stress changes in the prestressing steel are transmitted directly to the anchorage. Although most anchorages can develop the static strength of the prestressing steel, they are unlikely to develop its fatigue strength. Further, bending at an anchorage can cause higher local stresses than those calculated from the tensile pull in the prestressing steel. Bending is likely where the prestressing steel is connected to the member at a few locations only throughout its length or where there is angularity of the prestressing steel at the anchorage. Fatigue characteristics based on tests

*Personal communication from E. Schechter, Stressteel Corp., Wilkes-Barre, Pa.

Country	America	Japan								Switzerland		Britain
Anchorage Type	Conical Button Head	Button Head			Hammer Head				Swagged Nut	Conical Head		Conical Head
Mark	Series 3	B5	B7	B8	T5	T7A	T7B	T7C	E8	Button Heads	Conical	
Wire Diameter, in.	0.250	0.197	0.276	0.315	0.197	0.276	0.276	0.276	0.315	0.276	0.276	0.276 0.250
Radius, R / Diameter	—	0.25	0.29	0.28	0	0	0.49	0.89	—	—	—	—



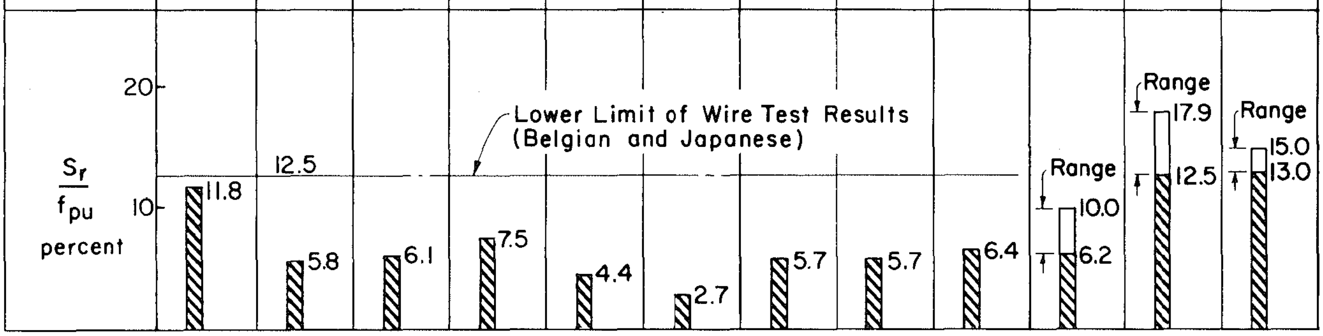


Fig. 15—Fatigue strength of anchorages at two million cycles

of single wire or strand anchorages are likely to overestimate the strength of multi-wire or multi-strand anchorages.

Tests on single wire anchorages have been conducted in the United States,[61] Great Britain,* Japan and Switzerland.[59] The types of anchorages tested and the results are shown in Fig. 15. In each case the ratio of the minimum stress to the nominal tensile strength of the wire was about 0.6. The broken line indicates the fatigue characteristics of the wire used in the Japanese tests, as estimated from the results of rotating beam tests. It corresponds also to the fatigue characteristics of the weakest wire in Fig. 11.

All anchorages shown in Fig. 15 developed the full strength of the wire for static loading. However, most resulted in a fatigue strength for the tendon of less than 50 percent of the fatigue strength of the wire. The exceptions are the conical anchorages for the Swiss, British, and American wires. If failures did not occur due to the fatigue loading, the static strength was not impaired. In the case of the American wire, five specimens out of seven took more than 10^7 cycles of the stress range shown without failure. The lowest life was 3.5×10^6 cycles for a specimen which failed at the button head fillets.

For the Swiss and British wires, ranges are shown on the bar charts in Fig. 15 to indicate the variation in results for different characteristics for the button head. The characteristics of a button head are influenced by the wire cutoff method, the type of heading equipment, the geometric characteristics of the head, the properties of the seating block, and the type of wire. Successive improvements have led to button heads showing no failures even after 10^7 cycles of a stress range equal to 0.13 times the tensile strength at an average of 0.6 times this strength. British tests on 0.276 in. (0.7 cm) diameter button-headed wires have shown that defects in the button head have little effect on the fatigue strength. For a wire with an ultimate tensile strength of 244 ksi (17,150 kgf/cm^2) tested at an average stress of 0.6 times that strength, the stress range for 2×10^6 cycles dropped from 0.15 times the tensile strength for a defect free head to a minimum of 0.12 times that strength for a diagonal split in the head. In contrast, a soft steel seating block for a defect free head resulted in a marked decrease in the fatigue life. The life dropped to 2×10^5 cycles for a stress range of 0.15 times the tensile strength, and the failure was due to fretting between the tendon and the soft steel.

The Japanese investigation showed that, to a limited extent, the strength increased as the ratio of the radius at the base of the head to the wire diameter increased. In these tests the fatigue

*Test reports supplied by A. H. Stubbs, Western Concrete Structures, Inc., Los Angeles, Calif.

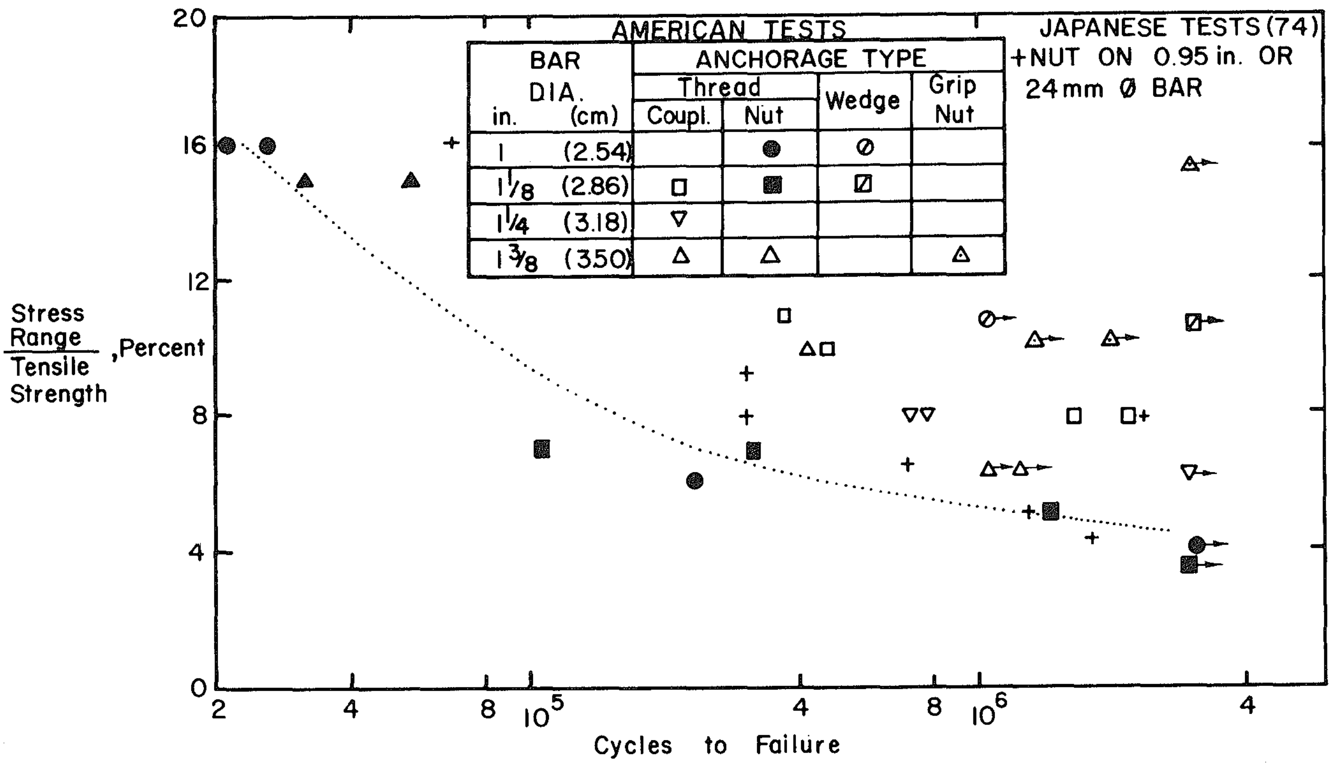


Fig. 16—Fatigue data for bar anchorages

crack usually developed where the shoulder for the head and the wire met. Clearly, the reduced fatigue capacity of the anchorage is due to the stress concentration caused by the change in section. The conically shaped anchorage forces the fatigue crack to develop at a section 50 to 80 percent larger in diameter than the wire.

Results for the fatigue tests conducted in the United States* and Japan[74] on anchorages for bars are shown in Fig. 16. Arrows indicate specimens for which failures did not occur. The dotted line is a lower bound to the test results. The ratio of the minimum stress to the tensile strength of the bar was about 0.6 for all tests. It is apparent that the stress range was insensitive to bar diameter or country of origin, and that all anchorages comply with the requirements of Section 7.2 of Reference 56. The reduction in the fatigue strength of the system for cut threads with couplers is less than for cut threads with nuts, and the reduction for both these systems is markedly more than for bars with grip nuts or wedges. In the American tests on grip nuts and wedges, a stress range of 0.1 times the tensile strength at a minimum stress of about 0.6 times that strength did not cause failure even after 3×10^6 cycles of loading.

Tests on single strand anchorages have been reported by several organizations.*,†,‡ For ½ in. (1.27 cm) seven-wire strand anchored in S7 and S9 C. C. L. spiral units† cast in small concrete blocks, failure did not occur within 1×10^6 cycles of a loading varying between 0.6 and 0.65 times the tensile strength of the strand. For ½ in. (1.27 cm), seven-wire strand anchored by 5½ x 2 in. (14 x 5 cm) cast steel anchors,‡ failures have not occurred within 0.5×10^6 cycles of loadings varying between 0.6 and 0.65, and between 0.56 and 0.64 times the tensile strength of the strand. Ten tests* on Stressteel S-H ½ in. (1.27 cm) Monostrand wedges have shown that for a 10 or 7 deg angle, this system can take without failure at least 5×10^5 cycles of a load varying between 0.6 and 0.66 times the strength of a 270 ksi (18,980 kgf/cm^2) seven-wire strand. For a load varying between 0.5 and 0.7 times the strength of the strand, failures occurred in the grips when one wire of the strand ruptured. Average fatigue lives were 57,100 and 54,700 cycles for 10 and 7 deg wedge angles. Results of foreign tests on proprietary anchorages for strand and multiple wire tendons are shown in Fig. 17. The sources of the data are indicated on the legend accompanying that figure. For all tests the minimum stress was about 0.56 of the tensile strength of the tendon. From a comparison of Fig. 17 and 13 it is apparent that anchorages for strand result in a fatigue strength of about 70 percent of the potential strength of the strand. The strength with a rope socket is only about 50 percent of the strength of the strand. For multiple wire anchorages it is ap-

*Personal communication from E. Schechter, Stressteel Corp., Wilkes-Barre, Pa.
†Test reports supplied by L. Gerber, The Prescon Corp., Corpus Christi, Tex.
‡Test reports supplied by K. B. Bondy, Atlas Prestressing Corp., Panorama City, Calif.

parent from a comparison of Fig. 17 and 11 that the reduction is of the same order as that for strand.

Several organizations in the United States have conducted tests on multiple wire or strand anchorages. A tendon* consisting of 90 one-quarter in. diameter, 240 ksi (16,870 kgf/cm^2) wires, anchored by button heads on an 8¾ in. (22.2 cm) diameter donut washer with fabrication blunders purposely incorporated in the washer, withstood, without failure, 55,100 cycles of a loading varying between 0.70 and 0.75 times the tensile strength of the wire. A tendon† consisting of nine ½ in. (1.27 cm) strands, anchored with three 3-strand S/H 10 deg wedges with the wedges on 1¼ in. (3.2 cm) radius at one end and 2¼ in. (5.7 cm) radius at the other end, withstood, without failure, 5×10^5 cycles of a load varying between 0.6 and 0.66 times the minimum guaranteed tensile strength of the tendon.

2.4.6 *Degree of bond*—Bond and cracking effects dominate differences between the fatigue characteristics of the prestressing steel in air and those of the same steel in a prestressed member. Prestressing steel can be characterized as unbonded, partially bonded, or well bonded to the concrete. When partially bonded, the steel is forced to follow the deformations of the member due to either the restrictions of the duct wall or the use of positioning devices. Cracking of the concrete usually develops at a maximum load intensity less than that resulting in a stress range in the steel sufficient for fatigue failure. The stronger the bond, the greater the local variation in stress in the steel due to cracking and the greater the effects of the distribution of the loading on the stress range.

There is contradictory evidence on whether prestressing steel has the same strength in air as it does in concrete. A statistically analyzable set of beam tests has yet to be conducted in conjunction with a comprehensive and statistically

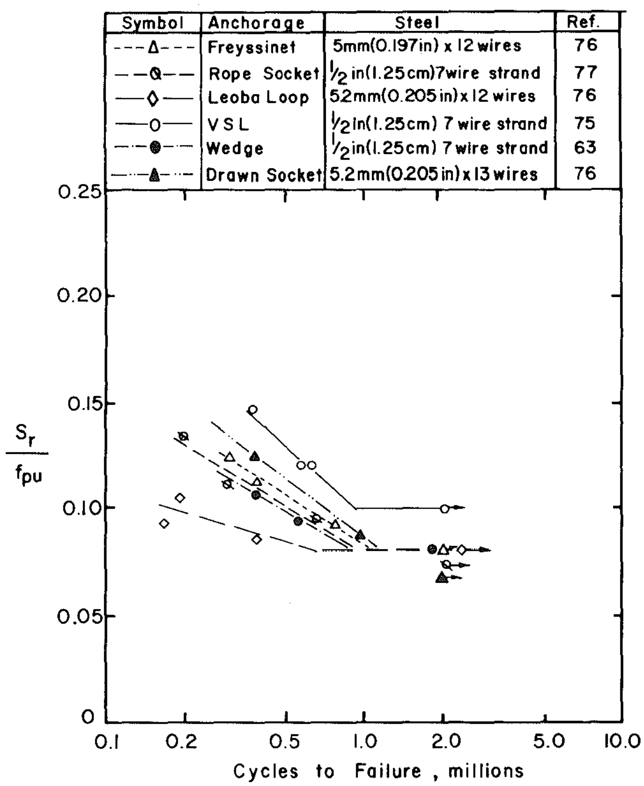


Fig. 17—Data for strand and multiple wire anchorages

analyzable test series in which the prestressing steel in the beam is stressed in air. In the most rigorous study to date,[32] close agreement was found between bond effects for 8 mm (0.315 in.) ribbed prestressing wires and those for reinforcing bars. For reinforcing bars, there was little difference in the fatigue strengths of embedded and nonembedded bars when the proportions of the transverse lug met bond requirements for static loading conditions. In contrast, for the poorer bond developed by prestressing wires, embedded wire had a fatigue strength about 15 percent lower than the strength of the wire in air. This reduction was attributed to effects caused by abrasion at the concrete-to-steel interface.

CHAPTER 3—FATIGUE OF BEAMS AND PAVEMENTS

Design of beams and pavements to resist fatigue is discussed in this chapter. Both reinforced and prestressed concrete beams are considered in Section 3.1. A summary of current design criteria relating to fatigue of pavements is presented in Section 3.2.

3.1—Beams‡

Beams designed in accord with the ACI Building Code[53] are generally proportioned to meet strength and serviceability requirements. In order to insure adequate performance at service load levels, beams subjected to repeated loads should be checked for the possibility of fatigue distress. Checking a design for safety in fatigue requires the following three steps:

1. Projection of a load histogram for the structural member;

*Test reports supplied by L. Gerber, The Prescon Corp., Corpus Christi, Tex.
†Personal communication from E. Schechter, Stressteel Corp., Wilkes-Barre, Pa.
‡Dr. Carl E. Ekberg, Jr., was the chairman of the subcommittee that prepared this section of the report.

2. Selection of locations where fatigue stresses may be critical; and

3. Determination of critical fatigue stresses and comparison of these stresses with permissible values.

Projection of a load histogram requires study of many factors relating to the nature of the repetitive loading, and is considered to be beyond the scope of this report. Furthermore, in many instances the matter may be inconsequential, because the question will be whether or not the concrete member is on the threshold of fatigue distress. Hence, it will be mainly important to project the number of cycles of maximum repeated loading which the member must resist during its design life.

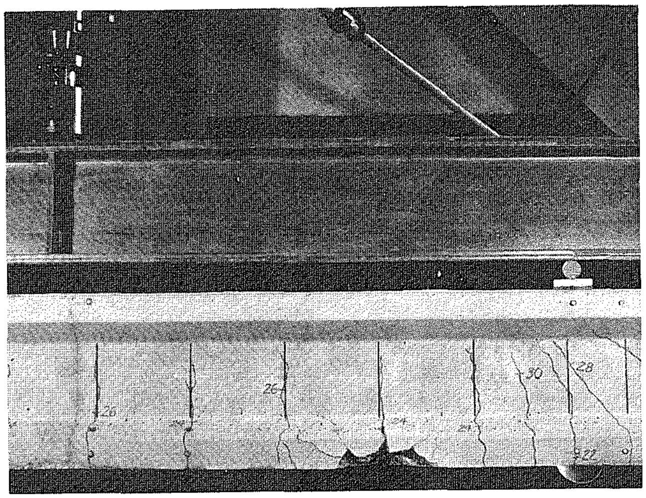
Fatigue distress may develop from excessive flexural, shear, or bond stresses. A view of a prestressed concrete I-beam in the process of a fatigue failure is shown in Fig. 18a. In this case, there are 5 strands in the bottom flange of the beam, 3 of which are in a lower layer. All of the wires in 2 of the 3 lower strands were fractured, and 3 in the 3rd strand, after 570,000 cycles of an "above-design" loading. The first evidence of damage was observed about 120,000 cycles before this point, when the flexural crack at this location began to widen noticeably.

Another similar I-beam that developed a shear fatigue failure after 400,000 cycles of an "above-design" loading is shown in Fig. 18b. Note that the dark vertical lines on the web indicate stirrups. All of the stirrups crossed by the shear crack have sustained fatigue fractures. More details on these tests are presented in Reference 78.

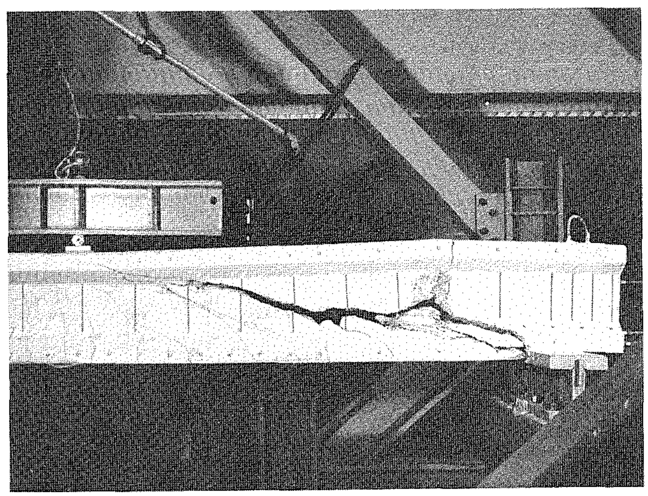
Any location where high stress ranges occur may be critical for fatigue. Locations of stress concentrations in steel reinforcement, such as at tendon anchorages or at points where auxiliary reinforcement is attached to deformed bar reinforcement by tack welding, are especially critical for fatigue. Bends in reinforcement may also be critical if they are located in regions of high stress.

Concrete is a notch insensitive material.[79] Hence, geometric discontinuities in the concrete due to holes or changes in section are not considered to affect its fatigue strength, although stress calculations must be based on the net section for large discontinuities.

Determination of critical fatigue stresses requires calculation of a minimum and maximum stress for specified loadings. In general, it is the stress range, which is the difference between the minimum and maximum stress, that is most critical for fatigue. Typically, the minimum stress is due to dead load, and the maximum stress is due to dead plus live load. Calculation of critical



a) Flexural



b) Shear

Fig. 18—Fatigue failures of a prestressed concrete I-beam

stresses is considered in more detail in the following sections on nonprestressed and prestressed members, as well as other special aspects which affect the behavior of these members.

3.1.1 *Nonprestressed members*—In this discussion, nonprestressed members are restricted to concrete beams reinforced with hot rolled deformed bars meeting the requirements of ASTM A 615-68.[43]

Flexural stresses in the concrete and reinforcement may be computed in accord with the provisions of Section 8.10.1 of ACI 318-71.[53] To determine if these stresses may possibly produce fatigue distress, the Committee recommends that the following criteria be used:

1. The stress range in concrete shall not exceed 40 percent of its compressive strength when the minimum stress is zero, or a linearly reduced stress range as the minimum stress is increased so that the permitted stress range is zero when the minimum stress is 0.75 f_c'.

2. The stress range in straight deformed reinforcement that may be imposed on minimum stress levels up to 40 percent of the yield strength shall not exceed 20,000 psi (1406 kgf/cm^2), or one-half of that amount in bent bars or bars to which auxiliary reinforcement has been tack welded.

Concrete is not believed to exhibit a fatigue endurance limit. The first criterion gives a conservative prediction of fatigue strength at a fatigue life of 10 million cycles. Deformed bar reinforcement does exhibit a fatigue limit. However, the second criterion is a conservative lower bound of all available test results on bars.

If the calculated fatigue stresses are higher than values indicated permissible by Criteria 1 or 2, the design should not necessarily be rejected. In these cases, evidence based on information in Sections 2.1 and 2.2 and elsewhere may provide a basis for allowing higher stresses.

Since most of the information included in Section 2.2 is based on fatigue tests of bars embedded in concrete beams, it is believed to be directly applicable to design. However, except for stress range, most of the variables which designers can readily control—bar size, type of beam, minimum stress, bar orientation, and grade of bar—do not have a large effect on fatigue strength. Other variables related to manufacturing and fabrication—deformation geometry, bending, and tack welding—are much more significant.

One factor not considered in Section 2.2 is that a structure is a composite of many members, each of which generally contain many reinforcing elements. As the results of the AASHO Road Test[20] indicated, fatigue fracture of one or more reinforcing elements does not necessarily result in failure of the structure. Rather there is evidence of distress due to increased deflections and wide cracks and hence there is opportunity to repair and strengthen the structure.

Recent unpublished research results at the University of Washington* indicate that special attention should be given to the shear fatigue strength of beams subjected to high nominal shearing stresses. Inclined cracking is a prerequisite for a shear fatigue failure. However, it is known that web shear cracks will form under repetitive loads at appreciably lower stresses than those assumed for static loading conditions.

For highly repetitive loading,[80] it is recommended that the range in nominal shear stress that is assumed to cause inclined cracking under a zero to maximum loading be taken as one-half the value of nominal shear stress carried by the concrete, v_c, specified in Section 11.4 of the ACI Code.[53] For other loadings, the range in nominal shear stress shall be linearly reduced from one-half of v_c to zero as the minimum stress is increased to v_c.

Where the nominal shear stress under service loads exceeds the values of v_c specified in Section 11.4 of the ACI Code, and the shear stress due to the repetitive live load plus impact exceeds 25 percent of the total nominal shear stress, it is further recommended that the shear carried by the concrete v_c be taken as zero for calculations of the required area of shear reinforcement. This recommendation will reduce the risk of a shear fatigue failure at bends in stirrup reinforcement.

3.1.2 *Prestressed members*—In this discussion, prestressed members are restricted to concrete beams reinforced with strand, wires, or bars that are prestressed to at least 40 percent of the tensile strength of the reinforcement. This reinforcement is presumed to meet the requirements of ASTM A 416-64,[64] A 421-65,[60] and A 322-64a,[81] respectively.

Whereas the determination of critical flexural stresses in nonprestressed members is relatively straightforward, the determination of critical flexural stresses in the concrete and tendons of prestressed members is quite complex. The reason is that flexural cracking must have occurred before fatigue of reinforcement can be critical. Hence an analysis which considers cracking must be employed.

Stress computations should be made using the basic assumptions given in Section 18.3 of the ACI Code,[53] although this procedure is quite tedious. A simplified method of analysis has been presented,[82,83] but the results may be too conservative to be useful. Other design alternatives have also been presented.[84,85,86]

As far as the fatigue strength of the concrete is concerned, the first criterion previously given in Section 3.1.1 is applicable. The Committee recommends that the following criteria be used for the prestressed reinforcement:

3. The stress range in prestressed reinforcement that may be imposed on minimum stress levels up to 60 percent of the tensile strength shall not exceed the following:

Strand and bars	$0.10\ f_{pu}$
Wires	$0.12\ f_{pu}$

Again, it may be possible to justify higher values of stress range by appropriate use of the data in Section 2.3 and elsewhere.[86] Also, results of recent research[78] indicate that if the nominal tensile stress in the precompressed tensile zone does not exceed $6\sqrt{f_c'}$, it may be assumed that fatigue of the prestressing reinforcement is not critical.

In prestressed members containing unbonded reinforcement, special attention shall be given to the possibility of fatigue in the anchorages or

*Personal communication from Dr. Neil M. Hawkins, University of Washington, Seattle, Wash.

couplers. Unbonded reinforcement is particularly vulnerable to fatigue if corrosive action occurs. Where information based on tests is not available, the fatigue strength of wire, strand, or bar anchorages shall not be taken greater than one-half of the fatigue strength of the prestressing steel. Lesser values shall be used for anchorages with multiple elements.

Most of the information included in Section 2.3 is based on fatigue tests of prestressing tendons in air. Concern has been expressed[87] over the applicability of the information to full sized members. Where comparisons[67,78] have been made, it was found that the observed life of test beams could be substantially less than that expected from *S-N* curves of the tendons alone. Differences were attributed to the difficulty of accurately determining stress in a tendon in a beam, and also to the local effects in the vicinity of a crack.

The need for statistical considerations in evaluating fatigue life of prestressed beams has also been cited.[67,88] Other information on the flexural fatigue behavior of large members[89-91] and bridges in the AASHO Road Test[92] is available.

Regarding the shear fatigue strength of prestressed concrete members, the discussion in Section 3.1.1 for nonprestressed members is also applicable to prestressed members. The mode of shear fatigue failure has been documented in recent research,[78,93] which demonstrated that prestressed beams have a remarkably high shear fatigue strength under very severe loading conditions.

3.2—Pavements*

Portland cement concrete pavements for airports and highways are subjected to repetitive loadings caused by traffic and cyclic environmental conditions. Although the resulting stresses may eventually cause cracking, localized distress does not necessarily terminate the pavement's useful life. Pavements normally are serviceable as long as load transfer across cracks and joints is effective, and the subgrade continues to support the slabs without excessive deflection. It is therefore necessary to design pavements to resist the expected repetitive traffic and environmental stresses for the predetermined service life.

Currently three types of concrete pavements are used in the United States: (a) plain pavements, with frequent joints and no reinforcement (with and without dowels); (b) reinforced concrete pavements, consisting of long slabs with distributed reinforcing and doweled joints;[94,95] and (c) continuously reinforced pavements (CRCP), consisting of very long slabs with more reinforcement than a reinforced concrete pavement and no transverse joints.[96]

Prestressed pavements may eventually be a fourth type. However, they are presently in a developmental stage. The majority of highway pavements are either of the plain or the reinforced concrete type. Hence, the following discussion will deal mainly with these types of pavements, although some of the comments will apply to the others.

Highway pavements are commonly designed by using either the Portland Cement Association (PCA) method,[97] or variations of the American Association of State Highway Officials (AASHO) method.[98] The PCA method is based on a modification of the Westergaard theory, and the AASHO method is based on the results of a comprehensive field study at the AASHO Road Test. For airports, the U. S. Corps of Engineers procedure is based on pavement performance and full-scale test track studies.[99]

The following is a brief description of some of the factors which affect the service life of concrete pavements.

1. *Traffic*—The volume and axle weights of the expected traffic must be predicted. For highways, these are predicted from highway department truck weight studies, and for airports they are based on aircraft manufacturers' data on the loads and configurations of existing and projected future aircraft.

2. *Environment*—Nonuniform stress gradients are created in pavement slabs because of restraint to slab movement induced by changes in temperature and moisture conditions. Temperature and moisture gradients also affect the performance of the slabs because they change the shape of the slabs and hence alter the degree of subgrade support.[100-102]

3. *Boundary conditions*—The stress state in the pavement is affected by subgrade friction, the type and efficiency of load transfer at joints, and the position of loads with respect to the joints and pavement edges.

4. *Support conditions*—Several phenomena may affect the underlying subgrade, and reduce the support which it provides to the concrete slab. These include loss of material by pumping, densification, and displacement of the subgrade, as well as soil volume changes due to moisture changes and frost.

In the following section, the PCA, AASHO, and Corps of Engineers methods are briefly reviewed. Other design methods are not specific in their evaluation of repeated loads. It is expected that the PCA, AASHO, and Corps of Engineers approaches will continue to be the basic models for design. Refinements in design methods are ex-

*Mr. Craig A. Ballinger was the chairman of the subcommittee that prepared this section of the report.

pected as more sophisticated analysis and computer techniques are developed.[103]

3.2.1 *PCA design method*—The PCA design procedure for highways is based on an extension of the Westergaard theory[104] which permits stress computations for multiple wheeled vehicles and relates support, axle load, and slab thickness to the stress created in the concrete. Only the heavy axle loads which stress the concrete to greater than 50 percent of its modulus of rupture are considered; i.e., the effects of passenger cars and light trucks are not considered significant. The criteria for the fatigue life of the pavement is the appearance of the first structural crack in the slab.

The basic tool of the designer using this method is a set of flexural design stress charts for highway vehicles and for aircraft. The charts are the result of analysis of exact wheel configurations involving influence charts[105] or computer programs.[106] Computed stresses are normalized by dividing by the design flexural strength of the concrete, and compared against a "standard" *S-N* curve to determine the allowable number of repetitions of load at each level. A percent damage is obtained by dividing the predicted number of loads by the number indicated to cause failure. These values are then accumulated in accordance with the Miner hypothesis, to determine whether the design life is satisfactory. The PCA method for airport pavement design[107] is similar to the highway design method.

3.2.2 *AASHO design method*—The philosophy associated with the AASHO design procedure is different than that of the PCA method, in that failure is considered to occur when pavement has deteriorated to a minimum tolerance level of serviceability.[108] Serviceability is a unique concept which is directly related to the pleasantness of ride experienced by the driver traveling over the roadway. The serviceability index of a pavement is affected by cracking, joint faulting, etc., only to the extent that it affects rider comfort. The serviceability index scale is linear from 5.0 down to 0.0. New pavements generally have an index between 4.2 and 4.6, and pavements are ready for resurfacing when the index drops to a value of 2.0 or 2.5 depending on the facility.

To apply this design method, all levels of axle loading are converted to equivalent 18 kip (8160 kg) single axle loads, by using a table of equivalency factors derived from the Road Test. As an example, the effect of one passage of an 18 kip axle load equates to 5000 repetitions of a 2 kip (907 kg) axle load. The thickness of the required pavement is determined directly by using a nomograph relating the thickness to the predicted number of equivalent axle loads to reach the minimum serviceability, the underlying subgrade support, and the allowable working stress in the concrete.

3.2.3 *Corps of Engineers method*—For this design procedure[99] load stresses are computed for the aircraft that are expected to use the pavement. Design charts indicate required pavement thicknesses for specific aircraft depending on concrete flexural strength, subgrade support and aircraft gear loads. The thickness so determined is for a fixed amount of traffic—5000 coverages of the design aircraft. The term "coverage" is used to convert the number of traffic operations to the number of full stress repetitions; i.e., a coverage occurs when each point of the pavement surface has been subjected to one maximum stress by the operating aircraft. An equation to convert operations to coverages considers the wheel configuration and transverse wander width of the aircraft passes on taxiways and runways. To recognize levels of traffic other than the fixed 5000 coverage level, the following increases in pavement thickness are specified; an increase of 5 percent for 10,000 coverages and up to 12 percent for 30,000 coverages.

NOTATION

f_c' = compressive strength of concrete

f_{pu} = ultimate strength of prestressing steel

f_r = modulus of rupture of concrete

n_r = number of cycles applied at a particular stress condition

N = fatigue life, i.e., number of cycles at which 50 percent of a group of specimens would be expected to have failed, or the number of cycles causing failure in a given specimen

N_r = number of cycles which will cause fatigue failure at the same stress condition as n_r

P = probability of failure

S = the stress calculated on the net section by simple theory such as $S = P/A$, Mc/I, or Tc/J without taking into account the variation in stress conditions caused by geometrical discontinuities

S_{max} = the stress having the highest algebraic value in the stress cycle, tensile stress being considered positive and compressive stress negative

S_{min} = the stress having the lowest algebraic value in the stress cycle, tensile stress being considered positive and compressive stress negative

S_r = stress range, i.e., the algebraic difference between the maximum and minimum stress in one cycle, $S_{max} - S_{min}$

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APPENDIX—SUMMARY OF SPECIFICATIONS RELATING TO FATIGUE*

A.1—Manual of Recommended Practice, American Railway Engineering Association; Chapter 8—Concrete Structures and Foundations; Part 17—Prestressed Concrete Structures; Section G—Repetitive Loads; Reapproved with revisions, 1969

A.1.1. *General*

The ultimate strength of concrete or steel subjected to repetitive loading may be less than static strength because of fatigue. Fatigue failure may occur in concrete, prestressing steel, anchorages, splices or bond.

A.1.2. *Concrete*

Fatigue strength of concrete in both tension and compression shall be considered to depend on the magnitude of stress, range of stress variation, and the number of loading cycles. Since high stresses and stress ranges are common, fatigue shall be considered when repetition of loading cycles of greater than design load may occur.

Fatigue failure is unlikely if the allowable stresses (after full prestress losses, of 0.4 f_c' compression and zero flexural tension) are not exceeded. If a large number of overloads is anticipated, a reduction in the safety factor may occur.

A.1.3. *Prestressing Steel*

Fatigue strength of prestressing steel shall be considered to depend upon magnitude and range of stress and the number of cycles of loading. Minimum stress is the effective prestress. Maximum stress and range of stress shall depend upon magnitude of live loads or overloads that may be repeated. Range of stress under service load will usually be small unless concrete is cracked. Cracking will be permitted only under temporary overload.

Devices for splicing steel may contain strain concentrations that lower fatigue strength. Consideration shall be given to fatigue whenever splices are used.

A.1.4. *Anchorages*

If steel is fully bonded, no difficulty should be expected in anchorage or end bearing as the result of repetitive loads. If unbonded steel subjected to repeated service loads or overloads transmitted directly to the anchorage is used, fatigue strength of the anchorage shall be given special consideration.

A.1.5. *Bond*

Failure of bond under repetitive loading shall be considered unlikely unless there is a significant number of repetitions of overload.

A.1.6. *Shear and Diagonal Tension*

Web reinforcement shall be provided as specified (provisions that essentially require sufficient web steel to force an ultimate flexural failure).

*Dr. William J. Venuti prepared this section of the report.

A.1.7. *Design Conditions*

Fatigue should not result in a reduction of strength if the following conditions are observed:

(a) Flexural compressive concrete stress shall not exceed 0.4 f_c' under either design load, or an overload that may be repeated many times, where f_c' is the compressive strength of concrete at 28 days.

(b) Tension shall not be permitted in concrete at critical cross section under either design load or overloads that may be repeated a number of times.

(c) Prestressing steel shall be bonded.

(d) Web reinforcement shall be provided as specified (provisions that essentially require sufficient web steel to force an ultimate flexural failure).

When these conditions cannot be followed, fatigue strength of all elements comprising the prestressed members shall be considered.

A.2—Building Code Requirements for Reinforced Concrete (ACI 318-71)

Chapter 18—Prestressed Concrete.

18.10—Repetitive loads

18.10.1—In unbonded construction subject to repetitive loads, special attention shall be given to the possibility of fatigue in the anchorages or couplers.

18.10.2—The possibility of inclined diagonal tension cracks forming under repetitive loading at appreciably smaller stresses than under static loading shall be taken into account in the design.

18.20—Post-tensioning anchorages and couplers

18.20.3—Anchor fittings for unbonded tendons shall be capable of transferring to the concrete a load equal to the capacity of the tendon under both static and cyclic loading conditions.

A.3—Strength and Serviceability Criteria, Reinforced Concrete Bridge Members (Ultimate Design); U. S. Department of Transportation, Federal Highway Administration, October 1969; Section 2.B—Serviceability at working loads

2.B.2 Fatigue considerations

(a) Concrete

The range of compressive stress in the concrete caused by a single passage of live load plus impact and centrifugal force, at working load level, shall be limited to 0.5 f_c' at points of contraflexure (concrete roadway slab excluded), and at sections where stress reversals occur. When the concrete is under repeated compressive stress at the same location, such as occur in hammer driven reinforced concrete piles, the range of compressive stress shall be preferably limited to values less than 0.75 f_c' for less than one thousand cycles of loading and limited to values less than 0.5 f_c' for two thousand cycles of loading or more.

(b) Reinforcement

The range of stress in straight reinforcement caused by a single passage of the live load plus impact at working load level, shall be limited to 20,000 psi. In bent bars the fatigue limit of the bend is considerably reduced. Bends in primary reinforcement shall be avoided at sections having a high range of stress.

A.4—Japanese National Railway Design and Engineering Code for Prestressed Concrete Railway Bridges

This code requires that both prestressing cables and anchorages be able to withstand one million cycles of a stress ranging between 0.6 times the guaranteed tensile strength of the cable and a stress 14.2 ksi higher.

A.5—The West German Code for Prestressed Concrete (DIN 4227)

This code requires manufacturers to specify the characteristics of their steel with reference to a Goodman diagram for values of the maximum stresses ranging between the service load stress and the 0.2 percent offset stress.

A.6—The West German Code for Reinforced Concrete (DIN 1045, 1972)

This code requires the manufacturer to prove the suitability of a particular reinforcing steel prior to its use in structures subjected to repeated loads. In addition the range of loading is limited to 1400 kgf/cm^2 (20,000 psi) for *BSt* 22/34 *G* (f_y = 31,000 psi) and to 1800 kgf/cm^2 (25,000 psi) for *BSt* 42/50 (f_y = 60,000 psi). The latter value is reduced to 1400 kgf/cm^2 (20,000 psi) at bends. For welded reinforcing mats the stress range is limited to 800 kp/cm^2 (11,000 psi).

This report was submitted to letter ballot of the committee which consists of 15 members; 14 members returned their ballots. Balloting was by sections and all sections received at least 11 affirmative votes; with a few "not voting" responses on individual sections, and one negative vote on Section 2.1.

ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction, and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be incorporated directly into the Project Documents.

ACI 221R-61

From ACI Journal, Nov. 1961

Selection and Use of Aggregates for Concrete

Reported by ACI Committee 621

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Presents available information on aggregates in four categories. (1) Evaluation of aggregate properties in terms of their influence on the properties of concrete. (2) Methods of determining aggregate properties and the limitations of these methods. (3) Features of aggregate preparation and handling which have a bearing on concrete quality and uniformity. (4) Selection of aggregate.

The report is limited to sand, gravel, crushed stone, and air-cooled blast-furnace slag aggregate. Lightweight aggregate and special heavy aggregate are not covered.

CONTENTS

INTRODUCTION

Notwithstanding that mineral aggregates represent the major volume of concrete—approximately 75 percent—the important role they serve as the principal ingredient is often overlooked because their cost is so much less than that of cement. Accordingly, Committee 621, Aggregates, was asked to prepare a report which will assist in the selection and use of aggregates for concrete.

It is the purpose of the committee that this report shall be a statement of information on aggregate that will assist in evaluating properties of aggregate in terms of their influence on properties of concrete, discuss features of preparation and handling which have a bearing on concrete quality and uniformity, and, in general, summarize what should concern the user about aggregate when he sets forth to do a first-class concrete job. The scope of the report is limited to sand, gravel, crushed stone, and air-cooled blast-furnace slag.* It is not intended to cover the use of lightweight aggregate or the special heavy aggregates.

This discussion is divided into four parts: (I) properties of concrete influenced by aggregate properties, (II) methods of determining aggregate properties and their limitations, (III) features of preparation and handling which have a bearing on concrete quality and uniformity, and (IV) selection of aggregate. Although the user does not specify the methods and equipment used in aggregate preparation or beneficiation, his requirements for aggregate possessing certain characteristics or properties will, in many cases, influence the processing of clean, sound aggregate of uniform quality. Part III is not intended as a guide for aggregate producers. It is included for the benefit of the user who frequently must do some handling of the aggregate.†

To the extent practicable, the selection of an aggregate should be based on knowledge of its significant properties as determined by service record, laboratory tests, and petrographic examination. Service record, when available in sufficient detail, is a most valuable aid in guiding judgment. To be significant, the record should cover structures with concrete proportions and exposures similar to those anticipated for the proposed work. Petrographic or other suitable procedures should be used to determine whether or not the aggregate in the structure and that proposed for use are sufficiently similar to make the service record significant. Further, it should not be overlooked that unfavorable service of old concrete without entrained air may not accurately indicate performance of concrete with the benefits of proper air entrainment.

A summary of the available information on mineral aggregates was published in 1948[1] and a summary of data on aggregate properties and their influence on the behavior of concrete is contained in "Significance of Tests and Properties of Concrete and Concrete Aggregates."[2] It contains nine papers on tests and properties of aggregates.

PART I — PROPERTIES OF CONCRETE INFLUENCED BY AGGREGATE PROPERTIES

Since aggregates occupy three-quarters of the volume of concrete, it is to be expected that properties of the aggregate have a major effect on the properties of concrete. Eight properties of concrete are discussed below, and the pertinent aggregate properties are considered.

Durability

For many exposures the most important property of concrete is its durability. There are several aspects to the problem and practically all of these are influenced by properties of the aggregate.

1. Resistance to freezing and thawing — Concrete containing good aggregate will not be resistant to freezing and thawing if the paste is inadequate; nor will concrete containing a frost-resistant paste if it contains unsound aggregate particles which are critically saturated. A particle is considered to be critically saturated when there is insufficient unfilled pore space to accommodate the expansion of water which accompanies freezing. The property of "soundness," usually defined as the ability of an aggregate to resist large or permanent changes of volume when subjected to freezing and thawing, heating and cooling, or wetting and drying, is related to the porosity, absorption, and pore structure of the aggregate. Rocks that can absorb water so as to become critically saturated are potentially vulnerable to freezing. Current laboratory studies and theoretical analysis suggest that for any rate of freezing there is a critical particle size, dependent on the perme-

*Definitions of blast-furnace slag, coarse aggregate, crushed gravel, crushed stone, fine aggregate, gravel, and sand are given in "Standard Definitions of Terms Relating to Concrete and Concrete Aggregates," ASTM C 125-58. Air-cooled slag is the product that results when molten slag is deposited in pits or banks for solidification under atmospheric conditions. It may then be further cooled by the application of water.

†A description of handling procedures is contained in (ACI 614-59), Reference 78.

ability and tensile strength, above which the particle will fail if critically saturated.[3] For fine-grained materials with low permeability the critical size may be in the range of normal aggregate sizes. For coarse-grained materials or materials with the capillary system interrupted by numerous macropores, the critical size might be so large so as to be of no consequence even though the absorption might be high. If potentially vulnerable aggregates are dry when used and are subjected to periodic drying in service, they may never become critically saturated. In this connection the paramount importance of an air-entrained paste in concrete exposed to frost action should be fully recognized.

When the aggregate contains only a few particles of unfavorable pore characteristics, freezing frequently produces, instead of general disintegration, the phenomenon known as "pop-outs" in which coarse aggregate particles near the surface push off the surface layer of mortar when they expand, leaving holes in the surface. Chert particles of low specific gravity, limestone containing clay, and shaly materials are well known for this behavior.

2. *Resistance to wetting and drying* — The influence of aggregate on durability of concrete subjected to wetting and drying is also controlled by pore structure of the aggregate. Although this problem is not nearly as serious as the problem of freezing and thawing, the differential swelling accompanying moisture gain of a material with a large amount of capillary absorption may be sufficient to cause failure of the surrounding paste. The amount of stress developed is proportional to the modulus of elasticity of the aggregate. In some cases pop-outs may occur.

3. *Resistance to heating and cooling* — Heating and cooling induce stresses in any material. If the temperature range is great enough, damage will result. For aggregates commonly used and for temperature changes ordinarily encountered, this is not a critical factor in concrete. However, in addition to this over-all effect, there has been theoretical speculation, and some laboratory evidence and field observations have been interpreted to support it,[4-8] that large differences between the values for paste and aggregate of the thermal coefficient of expansion or thermal diffusivity should produce significant stresses in concrete when subjected to normal temperature change. There is no general agreement that a problem exists here. In interpreting laboratory tests and field observations, it is difficult to isolate thermal effects from others such as moisture changes. Although the usual practice is not to restrict the expansion coefficient of aggregate for normal temperature exposure, aggregates with coefficients which are extremely high or low may require investigation before use in certain types of structures.

4. *Abrasion resistance* — Abrasion resistance is another property for which a high-quality well-cured paste is paramount. However, the hardness of the aggregate is an important factor and abrasion resistant floors and other concrete surfaces cannot be expected when soft aggregates are used.

5. *Alkali-aggregate reaction expansion* — Deterioration resulting from the alkali-aggregate reaction is recognized as a serious problem in certain geographic areas, principally in the southern and western parts of the United States. It may occur elsewhere in areas where the effect is concealed by the results of frost action. Reaction between alkalies in the cement, and certain siliceous constituents in some aggregates, produces harmful expansion. On the other hand, evidence of reaction has been observed in concretes in which no damage has occurred. The problem can be prevented by the use of a low-alkali cement or by the addition to the mixture of an adequate amount of a suitable pozzolan. The user confronted with the problem is referred to the extensive literature on the subject.[9]

6. *Fire resistance* — Concretes containing the aggregates within the scope of this report show little difference in fire resistance when exposed in a dry condition. The extensive field observations and laboratory tests[10-32] indicate a slight advantage for blast furnace slag, calcareous aggregates, and aggregates of igneous origin over siliceous aggregates of sedimentary or metamorphic origin.

7. *Acid resistance* — Acid resistance is more intimately associated with the cement paste than with the aggregates. However acid-resistant aggregates are required for special uses.[33]

Strength

Perhaps the second most important property of concrete, and the one for which values are most frequently specified, is strength. The types of strengths usually considered are compressive and flexural. Strength depends largely on the strength of the cement paste, and on the bond between the paste and aggregate. The strength of the aggregate also affects the strength of the concrete, but for many aggregates the differences are relatively small as compared to those resulting from differences in strength of the cement pastes in which they are used. Consideration of factors affecting the strength of the paste is beyond the scope of this report. The bond between the paste and aggregate tends to set an upper limit on the strength of concrete that can be obtained with a given set of materials, particularly in the case of flexural strength. Bond is influenced by the surface texture and cleanness of the aggregate. A rough-textured surface normally bonds better than a smooth surface. Coatings which adhere to the aggregate during mixing may interfere with bond. Many coatings have no deleterious effect. Those which are removed during mixing have the effect of augmenting the fines in the aggregates. Those which remain on the aggregate particle surface after mixing and placing have no particular effect unless they are of such a nature so as to interfere with bond or are of a chemical composition which will produce a deleterious reaction with alkalies in cement.[34] Clayey coatings will normally interfere with bond, while nonadherent dust coatings increase the water demand as a consequence of the increase in fines.[35]

While angular particles and those having rough, vesicular surfaces have a higher water requirement than rounded material, nevertheless crushed and uncrushed aggregates generally give substantially the same compressive strengths for a given cement factor. Some aggregates, which are otherwise suitable, have a higher than normal water requirement because of unfavorable grading characteristics or the presence of a large proportion of flat or elongated particles. With such materials it is necessary to use a higher than normal cement factor to avoid an excessively high water-cement ratio and as a result, insufficient strength. Water requirement may also be increased by nonadherent coatings and by poor abrasion resistance of the aggregate in that both increase the quantity of fines in the mixer.

There is experimental evidence[36] to show that at a fixed water-cement ratio strength decreases slightly as maximum size of aggregate increases particularly for sizes larger than 1½ in. However, for the same cement content, this apparent advantage of the smaller size may not be shown because of the offsetting effects of the increased quantity of mixing water required.

Shrinkage

The amount of shrinkage occurring during drying of concrete is dependent on the shrinkage potential of both the cement paste and aggregate, the total volume of paste in the concrete, and possibly the modulus of elasticity of the aggregate. Theoretical studies indicate that the higher the modulus of elasticity, the greater should be the restraint offered to the shrinking paste and hence the less the shrinkage measured in the concrete,[37,38] although the magnitude of the effect has not been determined experimentally. Since the quantity of paste depends on water requirement of the aggregate, such properties as maximum size, particle shape, grading, and cleanness are related to shrinkage. In addition, there is evidence that expansive clays, if present in aggregate, increase shrinkage to a greater extent than can be accounted for by increased water requirement.

Thermal properties

The coefficient of thermal expansion, specific heat, thermal conductivity, and thermal diffusivity of concrete are largely dependent on these same properties of the aggregate.

It has been demonstrated that the coefficient of thermal expansion can be computed approximately as the average of the values for the constituents weighted in proportion to the volumes present.[39,40] It has also been demonstrated that each of the materials composing the concrete contributes to the conductivity and specific heat of the product in proportion to the amount of the material present. Necessarily diffusivity is similarly affected.[41]

Unit weight

The unit weight of the concrete depends on the specific gravity of the aggregate, on the amount of air entrained, and on those properties discussed above which determine water requirement. Since the specific gravity of cement paste is less than that of normal aggregate, unit weight normally increases as the amount of paste decreases.

Modulus of elasticity

The modulus of elasticity of concrete is to some extent dependent on the modulus of elasticity and Poisson's ratio of the aggregate. However, for a given cement paste the modulus of elasticity of the aggregate has less effect on the modulus of elasticity of the concrete than can be accounted for by the volumetric proportions of aggregate in the concrete.[42]

Slipperiness

Slipperiness of pavements seems to be almost entirely related to the tendency of some aggregates to become polished as the concrete surface is worn by traffic.[43,44]

Economy

Economy of concrete is influenced by the quantity of cement necessary to produce required strength or other properties, by the availability or proximity of suitable material, and by the amount of processing required to produce suitable aggregate. Although well-shaped aggregates whether angular or rounded, graded within the limits of generally accepted specifications, will produce concretes of comparable quality at a given cement factor, it is pointed out in the report of ACI Committee 613[45] that with aggregates having characteristics which produce abnormally high water requirements, it is necessary to increase the cement content to maintain a fixed water-cement ratio.

The information presented in this section is summarized in Table 1.

PART II — METHODS OF DETERMINING AGGREGATE PROPERTIES AND THEIR LIMITATIONS

The aggregate properties listed in Table 1 are subject to either direct or indirect measurement in the laboratory. Some methods are com-

TABLE 1 — PROPERTIES OF CONCRETE INFLUENCED BY AGGREGATE PROPERTIES

Concrete property	Relevant aggregate property
Durability:	
Resistance to freezing and thawing	Soundness Porosity Pore structure Permeability Degree of saturation Tensile strength Texture and structure Presence of clay
Resistance to wetting and drying	Pore structure Modulus of elasticity
Resistance to heating and cooling	Coefficient of thermal expansion
Abrasion Resistance	Hardness
Alkali-aggregate reaction	Presence of particular siliceous constituents
Strength	Strength Surface texture Cleanness Particle shape Maximum size
Shrinkage	Modulus of elasticity Particle shape Grading Cleanness Maximum size Presence of clay
Coefficient of thermal expansion	Coefficient of thermal expansion Modulus of elasticity
Thermal conductivity	Thermal conductivity
Specific heat	Specific heat
Unit weight	Specific gravity Particle shape Grading Maximum size
Modulus of elasticity	Modulus of elasticity Poisson's ratio
Slipperiness	Tendency to polish
Economy	Particle shape Grading Maximum size Amount of processing required Availability

monly employed for specification purposes; others are not used for this purpose either because they require specialized equipment and techniques or because there is no general agreement on proper specification limits for the properties measured. Most emphasis here will be put on the specification tests. The others, while important for research, are not available to most concrete users.

Properties frequently specified or tests frequently performed

1. Particle size distribution, "Grading," "Gradation," "Sieve Analysis." (ASTM C 136, AASHO T27, ASA A37.8) — The particle size distribution has a major effect on the water requirement of concrete made from a given aggregate and thereby an effect on all the properties of concrete related to water requirement. It also has an important effect on the workability and finishing characteristics of fresh concrete.[46] The sieve analysis is probably the most frequently made of all aggregate tests. The particle size distribution is determined directly by passing samples of the aggregate through a nest of sieves of successively smaller openings and weighing the material retained on each sieve. This procedure also establishes the maximum size of particles present in the sample.

2. Specific gravity — An accurate knowledge of the specific gravity of both fine and coarse aggregate is required to calculate the batch weights needed to provide the desired absolute volumes of materials. Bulk specific gravity (saturated-surface-dry basis) is normally used in connection with concrete aggregates.

a. Coarse Aggregate (ASTM C 127, AASHO T85, ASA A37.5). The specific gravity of saturated surface-dry coarse aggregate is determined by weighing the material in air and in water.

b. Fine aggregate (ASTM C 128, AASHO T85, ASA 37.6). The method used for coarse aggregate does not lend itself to fine aggregate because of difficulty of containing the material for the weight in water determination. The procedure used instead is to place a known weight of saturated surface-dry sand into a vessel of known volume and to determine the volume of the material by measuring the amount of water required to fill the vessel.

3. Unit weight (ASTM C 29, AASHO T19, ASA A37.16) — The dry rodded unit weight, together with the specific gravity, provides a means for computing the voids in a unit volume of aggregate. In mixture proportioning it provides a means for estimating the amount of mortar required for a given coarse aggregate. The test is performed by determining the weight of aggregate required to fill a calibrated measure.

4. Absorption (ASTM C 127, C 128; AASHO T84, T85, ASA A37.5, A37.6) — While aggregate absorption data are frequently used as an aid in assessing the probable durability of concrete exposed to freezing and thawing, such a procedure must be applied with caution. Although most rocks with low absorption are durable, many durable rocks also have a high absorption. The structure of the pores as well as their total volume is important. A knowledge of absorption is essential to field control when surface moisture is determined by drying. Absorption is determined directly from the weights of a sample in the saturated surface-dry and oven-dry conditions.

5. Surface moisture — A knowledge of surface moisture is essential to field control of mixing water. Water carried into the mixer on the aggregate must be subtracted from the weight of added water while the scale settings for aggregate must be increased an equal amount. Surface moisture is commonly determined in the field by oven drying a sample and subtracting from the total water content the absorption as determined above. Devices are available in which chemicals added to a sample in a closed container react with water to produce a pressure which is a function of the amount of water present. Electrical devices for measuring the moisture instantaneously while the aggregate is in the weighing hopper also exist; these are based on resistance, dielectric, or neutron absorption principles. While none has been universally accepted as providing correct absolute values under all conditions, they are helpful in indicating when changes in surface moisture occur.

There is a standard test for a direct measurement of surface moisture of fine aggregate, ASTM C 70, AASHO T142, ASA A37.31. The amount of water that must be added to a sample to completely fill a calibrated vessel is used as an indication of the amount of water originally present on a sample of known weight or solid volume. The method is not in wide use because of the difficulty in removing all the air from the sample as water is added.

6. Soundness

a. Sulfate soundness test (ASTM C 88, AASHO T104, ASA A37.23). This widely used test consists of alternately immersing an aggregate sample in a solution of sodium or magnesium sulfate and drying it in an oven. The enlargement of salt crystals in the aggregate by rehydration during immersion after oven drying is presumed to simulate the increase in volume of water on freezing in the aggregate pores or cracks. Poor performance is indicated if, after test, a large part of the coarse aggregate sample will pass a sieve with openings 5/6 of those on which it was originally retained and if after test a large part of the sand sample will pass sieves on which it was originally retained. The choice of sodium or magnesium sulfate is largely one of individual preference. They do not yield results which are directly comparable.

Although specifications frequently contain acceptance limits, the sulfate soundness test has not been conspicuously successful in evaluating the resistance of aggregate to freezing and thawing in concrete. Its failure, apparently, is due not to the lack of similarity between sulfate crystal growth and ice formation but to the fact that in the sulfate test the aggregate is tested in the unconfined state. Unconfined freezing tests of aggregate particles have been no more successful than the sulfate test. Aggregate in concrete is surrounded by the fine-grained cement paste of extremely low permeability which greatly alters the exposure conditions.

b. Freezing and thawing of concrete. (ASTM C 290, C 291, C 292; AASHO T161). Freezing and thawing tests of aggregate in concrete probably provide the best measure of the soundness of aggregates, although no test has yet been standardized for general use. These methods permit the comparison of aggregates by subjecting air-entrained concrete containing samples of the aggregates to alternate cycles of freezing and thawing. Deterioration, if any, is measured by the progressive reduction in the dynamic modulus of elasticity. It may also be determined by periodic measurements of weight loss or length. The methods all require the use of moist-cured specimens, and they specify thawing in water. They differ in the speed of the cycle and in whether the freezing is done in water or air. The choice of a method is largely a matter of personal preference and available equipment. The use of accelerated freezing and thawing tests has been limited largely to within-laboratory comparison of aggregates. An evaluation of within-laboratory, between-laboratory and between-method reproducibility has been published by the Highway Research Board;[47] other data are given by Trudsϕ.[48]

Objections have been raised to certain aspects of these tests, and an alternate test[3] has been proposed and is in limited use. The chief objection to the usual tests is in the use of initially saturated specimens whereas most concrete in service is partially dried at the start of the winter. The alternate test requires that the specimens be in the same moisture condition as the prototype at the start of test. They are then soaked in water continuously and are periodically subjected to a single cycle of freezing to determine whether critical saturation has been attained. The latter situation is deemed to exist if the specimen dilates on freezing. If no component of the concrete is critically saturated, the specimen should contract on cooling even through the freezing point. Concrete which can withstand soaking for a time equal to the freezing season without becoming critically saturated is considered immune to frost action in the particular environment under consideration.

7. *Abrasion resistance (coarse aggregate)* — Tests for abrasion resistance measure degradation caused by a combination of impact and surface abrasion. The test provides an indication of probable breakage in handling, stockpiling, and mixing. It is widely used as an index of aggregate quality and provides some indication of strength-producing potential. Although many specifications contain numerical limits, it should be noted that minerals differ in their relationship between abrasion resist-

ance and strength, and crushed material abrades differently than rounded aggregate.

a. Los Angeles (rattler) machine (ASTM C 131, AASHO T96, ASA A37.7)

b. Deval machine (ASTM D 289)

Both these tests evaluate abrasion resistance from the increase in fineness produced by tumbling the aggregate with steel balls inside a steel vessel. The Los Angeles apparatus is by far the more widely used. It is a more discriminating test and requires less sample preparation than the Deval machine. By determining percentage of wear on a single sample after two different periods of exposure the Los Angeles rattler can be used to detect the presence in the sample of constituents that are markedly nonresistant to abrasion. A uniform material yields percentage of wear at a uniform rate whereas a sample containing a component that is markedly nonresistant to abrasion yields percentage of wear rapidly at first but with a diminishing rate as the test progresses.[49]

8. *Cleanness and deleterious substances*

a. Amount of material finer than the No. 200 sieve (ASTM C 117, ASA A37.4, AASHO T11). In this test the coarse aggregate is washed vigorously in water, and the wash water containing the fine material in suspension is passed over a No. 200 sieve.

b. Test for clay lumps in coarse aggregate (ASTM C 142, ASA 3728, AASHO T112). Material which can be pulverized by the fingers is hand pulverized and separated from the sample by sieving.

c. Test for lightweight pieces (ASTM C 123). Aggregate deposits in which poor performance is associated with a minor lightweight fraction are usable when the light particles are removed by beneficiation or are present in small quantities. A limit is frequently placed on the permissible amount of light material when such deposits are used. To test for the presence and amount of such particles the sample is placed in a liquid having a specific gravity between that of the predominant component of the aggregate and the lightweight constituents whose presence is to be detected. The light materials float on the liquid and can be removed.

d. Test for organic impurities in sand (ASTM C 40, ASA A37.19). Organic materials may interfere with normal cement hydration. Specifications usually require that fine aggregate be free of injurious amounts of organic impurities. To perform the test the sample is placed in a sodium hydroxide solution and after 24 hr the color of the supernatant liquid is observed.

e. Sand equivalent test, test of the California Division of Highways. A sample of sand is agitated in a weak calcium chloride solution and the relative volumes of sand and flocculated clay determined. The test provides information on both the amount and the activity of the clay.

f. Alkali-aggregate reactivity. This is a special problem which does not influence specifications except in those areas where the problem is known to exist. In recognizing the problem, the service record of concrete made from any particular aggregate is especially important. If this service record shows cases of excessive cracking, or if no service record is available, it is suggested that a petrographic examination be made of the aggregate. The following laboratory tests are available.

(1) Quick chemical test (ASTM C 289). This has the advantage that it can be performed in 3 days, but for many aggregates the results are not conclusive.

(2) Mortar bar test (ASTM C 227). This test, in which the expansion of mortar bars stored over water is measured, is more conclusive but has

the disadvantage of requiring several months and of requiring that coarse aggregate be crushed rather than tested in its normal state. With larger specimens, however, uncrushed aggregate may be tested.[50]

Hardness (coarse aggregate)

a. Scratch test (ASTM C 235). This test, intended to identify materials that are soft, including those which are so poorly bonded that the separate particles in the piece are easily detached from the mass, occurs frequently in specifications. Particles fail if they are scratched by a pointed brass rod of specified properties under a specified load.

b. Dorry hardness machine.[51] A rock cylinder is subjected to surface wear by finely crushed quartz on a revolving metal table.

c. Shore scleroscope.[52] Hardness is determined by the rebound of a diamond-tipped hammer dropped vertically on the test surface. This test is not widely used for mineral aggregates.

Properties not normally specified or tests infrequently performed

Most of the following tests have not been standardized.

1. *Toughness* — Falling hammer impact test.

   a. Rock cylinders (ASTM D 3, ASA A37.73, AASHO T5-35).

   b. Individual pieces of aggregate.[53]

2. *Compressive strength*

   a. Cylinder or prism compressive tests.[54]

   b. Crushing loose aggregate in a steel cylinder,[55] British Standard 812.

3. *Modulus of elasticity and Poisson's ratio*

   a. Compressometer tests of cylinders or prisms.[56]

   b. Tests of cylinder or prisms in which aggregate particles are included in a matrix of known properties.

4. *Particle shape*[34]

   a. Determination of dimensional ratios by proportional calipers, U.S. Corps of Engineers CRD C 119.

   b. Determination of void content.[57]

5. *Surface texture*[34] — Replica surface analyzer.[58]

6. *Porosity*

   a. Calculated from true and bulk specific gravities (ASTM C 127, C 128; ASA 27.5; AASHO T84, 785).

   b. Porosimeter.[59]

7. *Pore structure*

   a. Microscopic.[60]

   b. Adsorption.[61]

   c. Porosimeter.[62]

8. *Permeability* — Direct measurement of liquid or gas flowing through a specimen.[63]

9. *Specific heat* — Method of mixtures, U.S. Corps of Engineers CRD C-124.

10. *Thermal diffusivity*

    a. Calculated from specific heat, density, and conductivity.[41]

    b. Heating a solid block and measuring surface and interior temperatures.[64]

11. *Coefficient of thermal expansion*

    a. Optical lever extensometer.[4]

    b. Resistance strain gage mounted on coarse aggregate. (U.S. Corps of Engineers CRD C 125).

    c. Dilatometer.[65]

    d. Mortar bar for fine aggregate (U.S. Corps of Engineers CRD C 126).

Visual observation and petrographic examination (ASTM C 295)

Visual observation can be a valuable addition to laboratory testing. A geologist can identify types of rock, formations in which they occur, and possible sources of trouble in a deposit. Even one not trained in geology can learn much by observing laboratory test specimens at the conclusion of tests such as those for compressive and flexural strength, soundness, and resistance to freezing and thawing.

A logical extension of this basic visual process, on which increasing emphasis is being placed in selecting concrete materials, is petrographic examination. The latter is defined as visual examination and analysis of the material in terms of both lithology and properties of the individual particles.[66,67] The procedure commonly uses a hand lens and petrographic and stereoscopic microscopes, but it can employ other techniques such as x-ray diffraction and differential thermal analysis.

Petrographic examination can quickly supply much pertinent information on the properties mentioned in preceding sections. It also assists in the interpretation of tests. It is the best available method for determining the presence of aggregate constituents capable of deleterious alkali reactivity and those containing swelling clays; it is also a good method for classifying texture (grain size) and structure (grain interlock) in aggregate constituents. Because of the nature of the examination, sample sizes are necessarily small. A large amount of work, therefore, is required for quantitative analysis of high precision. Much of the information is obtainable in an entirely objective fashion. Some of it, however, requires more personal interpretation than is required for most laboratory tests.

Petrographic studies commonly consist of two parts: (1) identification of the material or component; (2) an attempt to predict performance from past records of similar materials in service or from theoretical considerations. The method has found useful application in large organizations and has great potential, especially as an integral part of a well-equipped laboratory. By petrographic examination, valuable lessons of the past can be applied more tangibly to current problems provided that the pertinent data were originally well documented. However, there is a great need for more good service records of specifically identified materials under various conditions. Service records on materials of apparent similarity are not, however, necessarily dependable indices of performance unless restricted to the same geological areas.[68] In any case, long experience is required to be able to interpret, in terms of service, data obtained by petrographic examination. It should be emphasized that specialized experience on the part of the petrographer is mandatory and close liaison with the concrete technologist desirable.

PART III — FEATURES OF PROCESSING AND HANDLING WHICH HAVE A BEARING ON CONCRETE QUALITY AND UNIFORMITY

Basic physical and chemical characteristics of aggregate cannot be altered by processing, although the quantities of certain deleterious particles can be reduced. Preparation and handling affect such important aggregate properties as gradation; uniformity of moisture content; cleanness; and, in the case of crushed aggregate, particle shape; thereby having an important influence on concrete quality. Economic factors will determine the degree to which processing can be carried in an effort to achieve desirable properties. Attention is directed to the appended list of references to the literature.

Aggregate processing may be divided into two broad classifications: (1) basic processing to achieve, principally, suitable gradation and cleanness; (2) beneficiation, to remove deleterious constituents.

Basic processing

Processes typically employed to provide aggregate of satisfactory grading and cleanness include the following:

1. Crushing and grinding — Stone, slag, and large gravel require crushing to provide the required distribution of sizes. Grinding is sometimes employed to produce sand sizes. In crushing gravel to produce angularity, rather than for necessary size reduction, quantities of desired large or intermediate fractions may be reduced and amounts of finer sizes unduly increased. There is also the possibility of creating undesirable particle shapes which may detract from workability or increase water requirement of concrete. As discussed below, crushing may have a by-product advantage for some sources by reducing the amount of soft and friable particles.

2. Screening — Screening is the primary process for producing desired gradation in the coarse aggregate size range. Although dry screening of crushed stone and blast furnace slag is quite common, it may be necessary at times to apply water during the screening process to remove fine particles which may be present in the material. In the case of gravel, screening is usually accompanied by the application of water to wash the material and expedite the separation of sand from coarse sizes. Normally, screening is employed only on sizes larger than the No. 8 sieve, although there are exceptions.

3. Washing — Washing is done to remove silt, clay, and excess fine sand. It usually begins with application of water during screening and is completed with removal of the unwanted fines in the overflow from water classification. If the aggregate contains clay, mud, mudballs, or organic impurities in such quantities or so firmly attached that ordinary washing will not clean it adequately, scrubbers or log-washers may be needed. Taggart[69] describes the available processes.

4. *Water classification*—Sizing and control of gradation in the finer sizes are usually accomplished by classification in water. A wide variety of classifying devices are used for this purpose, all of which are based on different settling rates of different sized particles. Water classification is not feasible for sizes larger than about ¼ in. Sizing is not so sharp as in screening. However, the gradation can be controlled with considerable accuracy by suitable reblending, in spite of the overlapping in sizes.

Beneficiation

"Beneficiation" is a term used in the mining industry to describe the improvement in quality of a material by the removal of unwanted fractions. Success of a process depends on significant differences in the physical properties of desirable and undesirable constituents, such as hardness, density, and elasticity. The method to be employed, therefore, if any will be practicable, depends on the nature of the individual deposit. Processes which have been used with variable amounts of success include the following.

1. *Crushing*—Crushing may be used to reduce the quantities of soft and friable particles in coarse aggregates. Certain impact crushers are particularly adaptable to "selective" crushing. The degraded material is eliminated either by screening or by classification. The costs of installation and operation are likely to be high and there is always loss of sound material, which is frequently excessive; and removal of the degraded fractions may be difficult or expensive. On the other hand, in the case of many deposits, selective crushing is the only process available to make the material suitable for use.

2. *Specific gravity separation*—In many deposits, the deleterious fractions are of significantly lower specific gravity than the better quality parent material. Advantage is taken of this characteristic in several beneficiation processes.

a. High velocity water and air. Light materials, such as wood, miscellaneous trash, and some lignite, may be removed in a rapidly moving stream of water which carries the floatable material while allowing the heavier aggregate to sink. Some "trash" removers employ a vortex of water which throws the heavy particles to the periphery while allowing the light materials to rise for removal from the surface. Only large differences in specific gravity will yield efficient separation. High velocity air has also been used for this purpose in a few instances.

b. Jigs. They may be used to separate materials with much smaller differences in specific gravity than that required for high velocity water separation. Lightweight shales and cherts are prominent examples of materials which can be removed with good success. A jig is essentially a box with a perforated bottom in which a separating layer is formed by a pulsating water current. In certain types of jigs, even the finest sizes of coarse aggregate may be treated effectively. The overlap in specific gravity range common to both the rejected and accepted fractions can be maintained at a reasonably low figure where the procedure is applicable. Taggart[69] describes a variety of jigs and their operation (see also References 70, 71, and 72).

c. Heavy media separation. Water suspensions of heavy minerals may be used to separate materials of different specific gravities—the difference required being substantially less than required for successful jigging. The heavier fractions sink and the lighter material floats. The medium commonly used is a suspension in water of magnetite or magnetic ferrosilicon, substances which are susceptible to recovery by magnetic separators for re-use. By close control of the suspension, minimizing of turbulence, and elimination of fine particles (below about the No. 8 sieve), accurate elimination of material below a selected specific gravity level can be made. Removal of unwanted material of high specific gravity could also be accomplished by this process, although the need for such a procedure is rare. As in the case of other beneficiation methods, the practical applicability of heavy media separation depends on the nature of the parent aggregate, the degree to which specific gravity distinguishes between good and bad particles, and the requirements of the concrete for which the aggregate is intended.[70,72,73]

3. *Elastic fractionation*—Elastic fractionation is a recently developed process of extremely limited applicability. It involves dropping the aggregate particles onto a steel plate from which those with higher modulus of elasticity bounce farther than the presumably less desirable particles of lower elasticity. A separation is achieved by collecting the portions of differing rebound in separate compartments. There are several serious limitations to the method: (1) the shape of all particles must be somewhat similar, so as not to unduly affect rebound; (2) the process is not applicable to crushed aggregates because sharp edges interfere with elastic rebound; and (3) certain deleterious rocks, such as some of the harmful varieties of chert, have good elastic properties and are not eliminated. Elastic fractionation has been found to be applicable in only a few cases. As of the date of preparation of this report the committee knows of only one full-scale installation—and that is coupled with heavy media separation.[73]

4. *Magnetic separation*—The removal of iron from blast furnace slag with electromagnets is highly effective when the iron is in metallic form and has been liberated from the surrounding slag by crushing or grinding.

For general information on processing, the reader is referred to papers by Goldbeck,[74] Hubbard,[75] and Walker[76] and to the bibliographies contained in those papers. The art of processing is developing rapidly and current articles in the trade press keep abreast of improvements. Detailed information on many of these processes will be found in *Handbook of Mineral Dressing* by Taggart.[69] A general discussion of production and manufacture of aggregates is given by Rockwood.[77]

Handling of aggregates

Much can be done in the handling of aggregates to increase the likelihood of good performance. Conversely, basically good material may yield inferior results due to abuse in handling. Procedures for maintaining uniformity of gradation and moisture content are discussed in Section II of "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-59)."[78] Fig. 1 from that report is reproduced here and the principal recommendations which pertain to aggregates are given in abbreviated form.

1. Segregation in coarse aggregate is minimized when it is separated into appropriate individual size fractions to be batched separately.
2. Undersize smaller than the designated minimum size in each fraction should be held to a practical minimum, always less than the amount permitted by the specifications, and should be uniform in amount, particularly in the smaller sizes.
3. Stockpiles should be built in horizontal or gently sloping layers. Conical stockpiles or any unloading procedure involving the dumping of aggregates down sloping sides of piles should be avoided.
4. Trucks and bulldozers should be kept off stockpiles as they cause breakage and contamination.
5. Effective measures should be taken to insure accurate separation of sizes within specification limits as the aggregate is deposited in the batch plant. This can be accomplished by finish screening or by careful attention to stockpiling practices such as providing adequate separation between stockpiles and operating cranes to avoid swinging buckets of one aggregate over another.
6. Storage bins should have the smallest practicable horizontal cross section; the bottoms should slope at an angle not less than 50 deg from the horizontal toward a center outlet; and they should be filled by material falling vertically over the outlet.
7. Two sizes of sand cannot be blended satisfactorily by placing them alternately in stockpiles, cars, or trucks. Blending, if required to improve grading, should be done by feeding the different sizes into a common stream from regulating feeders onto the belt or loader. Where two or more sizes of sand are employed, it is preferable to batch them separately.
8. Wind should not be permitted to segregate dry sand.
9. To the extent practicable, wet sand should be drained until it reaches a uniform moisture content. Generally a satisfactory and uniform stable condition will be reached in about 48 hr or less.

Control of particle shape

The particle shape of crushed aggregates is dependent on the crushing equipment used. A universal guide cannot be written because it has been found from experience that equipment which produces acceptable particle shape with one type of rock will not necessarily produce acceptable shape with another type. This is a problem for the producer which is mentioned here so that the user may be aware of it. Normally he will be able to observe the particle shape before he accepts the aggregate. In some cases, particularly on large projects, the user may be in a position to do some experimenting with the material to determine in advance what sort of particle shape can be feasibly produced.

PART IV — SELECTING AGGREGATES

A high quality aggregate consists of particles which are free from fractures, not easily abraded, favorably graded, and not flat or elongated;

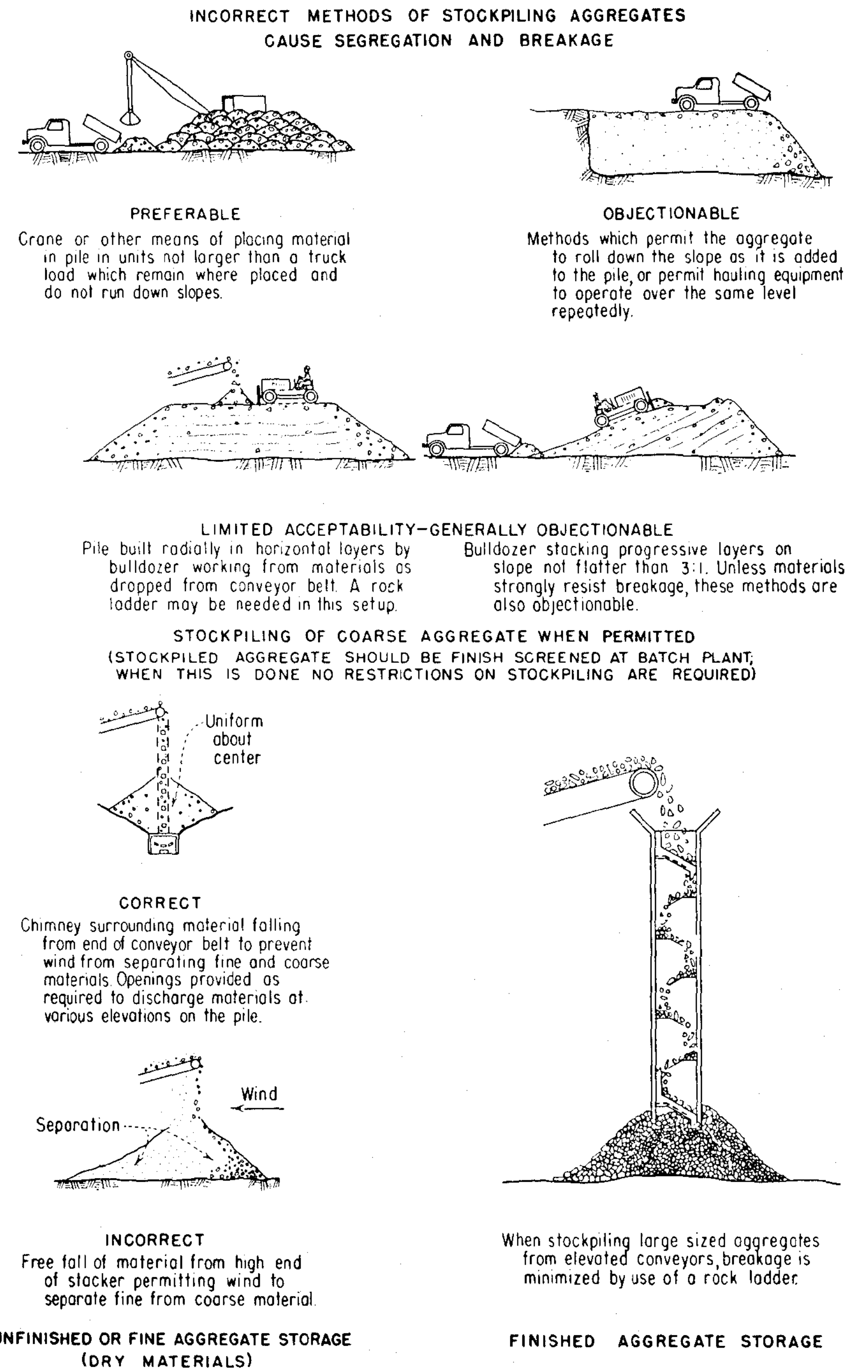

Fig. 1—Correct and incorrect handling of Aggregates. (Reproduced from ACI 614-59, Reference 78)

which do not slake when wetted and dried, whose surface texture is relatively rough with little or no unfavorable capillary absorption and which contain no minerals that interfere with cement hydration or react with cement hydration products to produce excessive expansion. Other properties are not so easily specified; for instance, thermal conductivity should be high if the chief concern is getting the heat out of the interior of a dam, and low in a building wall where insulating value is important.

The ideal aggregate is seldom available. The problem is to decide the level of performance that is required in a given situation and determine the degree to which it is economically attainable. It is necessary to appraise the available aggregates. All tests have limitations which make them not completely reliable; therefore, the service record, if available and properly interpreted, becomes a valuable source of information.

Since the causes of deterioration of concrete are many and even experts are often in disagreement in a given instance, there is a great risk of wrongfully condemning an aggregate on the basis of its presence in defective concrete. Records must be dependable and the evidence of a recurring nature before rejection is made. However, popouts provide a dependable criterion of identifying an undesirable component in an aggregate. A structure completely sound after 10 years or more representative service can be assumed to constitute an "endorsement" of all materials used in it, including the aggregates. There is the possibility, however, that potentially reactive aggregates may have a satisfactory service record if they have been used only with low-alkali cements. Furthermore, it is important to check that the quality of the material currently available from the source is at least equal to that used in the structure of good record. Where deterioration is associated with a minor constitutent, beneficiation may make a previously undesirable aggregate usable. Close visual observation is an all-important aid to good judgment. Where the service of a petrographer is available, the appraisal of an aggregate's service record can be made on a much more scientific basis. The petrographic examination provides information on a wide variety of properties and frequently makes it possible to compare samples from a newly developed aggregate source with others of known service record.

In selecting an aggregate it is economical to require only those properties pertinent to its use in a particular project. As a general guide the following criteria are suggested.

a. Regardless of use, the grading of the aggregate should be uniform throughout its period of use and should conform to some reasonable grading requirements. Excellent concrete can be made with aggregates differing quite widely in grading characteristics so long as they remain within the tolerances of usual specifications, for example, ASTM C 33. Actually, good concrete can be made with some aggregates outside these limits, including those showing discontinuous gradings, if enough care is taken in proportioning concrete mixtures[45] to determine optimum proportions. However, unless the job is large enough to justify the experimentation needed to establish effective mix proportions, or the grading of the particular aggregate is known to produce satisfactory concrete, aggregates meeting standard grading specifications should be used. In any case, it is important that once the grading is established, it should be maintained constant within rather close tolerances. Otherwise effective job control is impossible.

If after establishing optimum proportions for fine aggregate, coarse aggregate, and cement, the fineness of the sand or the percentage of undersize in the coarse aggregate increase, the water requirement for the required slump will be raised. If water is added without compensating adjustments in cement, strength and durability will be reduced. A decrease in the fineness of the sand can adversely affect the workability of the concrete by introducing harshness. Fluctuations in fineness also introduce an added difficulty in achieving uniform air entrainment. The abrasion resistance of the aggregate influences the stability of the gradation during handling and mixing.

b. An aggregate with unfavorable particle shape should not necessarily be rejected in favor of a more expensive aggregate with better particle shape if the cost of additional cement required for the first aggregate is less than the extra cost of the second aggregate, and the use of the additional cement will not be detrimental.

c. An aggregate that is contaminated with organic material to such an extent that the contamination interferes materially with the setting of the cement should not be used.

d. An aggregate that will not produce concrete of the required strength should not be used. If required strength can be achieved only with an excessively high cement factor, use of the aggregate is probably uneconomical.

e. A material to be used in concrete to be exposed to severe freezing and thawing should be shown to be capable of producing frost resistant concrete either by service record or freezing-and-thawing tests.

f. A material to be used in concrete exposed to severe weathering that should also maintain a defect-free appearance should be essentially free of particles that are soft or friable, that have unfavorable capillary absorption, or that will weather to produce staining.

g. A material containing or consisting of substances that could react with alkalies in the cement to cause excessive expansion should not be approved for use in concrete that will be exposed to wetting unless it is also required that low-alkali cement is used or that an adequate quantity of a suitable pozzolan is used, or both.

h. In unusual circumstances the aggregate user may wish to obtain materials with particular thermal or elastic properties; in such cases he may expect to pay a premium for the aggregate.

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ANSI/ACI 223-77

ACI Standard

Recommended Practice for the Use of Shrinkage-Compensating Concrete (ACI 223-77)*

Reported by ACI Committee 223

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Shrinkage-compensating concrete is used extensively in various types of construction to eliminate or minimize cracking caused by drying shrinkage. Although its characteristics are in most respects similar to those of portland cement concrete, the materials, selecting of proportions, placement, and curing must be such that sufficient expansion is obtained to compensate for subsequent drying shrinkage. This recommended practice sets forth the criteria and practices necessary to insure that expansion occurs at the time and in the amount required. In addition to a discussion of the basic principles, methods and details are given covering structural design, concrete mix proportioning, placement, finishing, and curing. A bibliography of the major references covering expansive cements and concretes is also appended.

Keywords: admixtures; aggregates; calcium aluminates; **concrete construction**; concrete finishing (fresh concrete); concretes; consistency; curing; drying shrinkage; ettringite; **expansive cement concretes**; expansive cement Type K; expansive cement Type M; expansive cement Type S; **expansive cements**; formwork (construction); gypsum; hydration; joints (junctions); **mix proportioning**; placing; restraints; **shrinkage compensating cements**; **shrinkage compensating concretes**; **structural design**; tests.

*Adopted as a standard of the American Concrete Institute in March 1977, in accordance with the Institute's standardization procedure.

†Deceased.

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CONTENTS

CHAPTER 1—INTRODUCTION

1.1—Background

In the report "Expansive Cement Concretes—Present State of Knowledge,"[1] ACI Committee 223 described research investigations and structures which used shrinkage-compensating cement. It outlined the theory and results of studies into the general behavior of such cements, the effect of concrete materials, mixing and curing conditions, admixtures, etc., as well as the success of early field applications. The recommended practice which follows summarizes the theoretical aspects of the previous report and recommends design and field practice for the utilization of shrinkage-compensating cements and concretes.

1.2—Purpose of shrinkage-compensating concrete

Shrinkage-compensating concrete is an expansive cement concrete which is used to minimize cracking in concrete pavements and structures caused by drying shrinkage. Drying shrinkage is defined as the contraction which is caused by moisture loss from the hardened concrete. It is exclusive of plastic volume changes which occur when surface evaporation exceeds the concrete bleeding rate and other length of volume changes induced by temperature, structural loads, or chemical reactions.

The amount of drying shrinkage which occurs in concrete depends on the characteristics of the materials, mix proportions, placing methods, and curing. When a pavement, floor slab, or structural member is restrained by subgrade friction, reinforcement, or other portions of the structure during drying shrinkage, tensile stresses develop. While structural portland cement concretes normally possess tensile strengths in the range of 300 to 800 psi (2100-5500 kPa), drying shrinkage stresses are often large enough to exceed this strength resulting in cracking. Furthermore, because of the probable existence of additional stresses imposed by loads, temperature changes, settlement, etc., the inherent tensile strength of the concrete cannot be relied on to resist shrinkage stresses. The frequency and size of cracks which develop in many structures, therefore, depend on the amount of shrinkage and restraint.

Shrinkage-compensating concrete is constituted and proportioned such that the concrete will increase in volume after setting and during hardening. When properly restrained by reinforcement or other means, concrete compressive stresses are induced. On subsequent drying, the shrinkage so produced, instead of causing a tensile stress to develop which might result in cracking, merely relieves the compressive stress caused by the initial expansion.

1.3—Scope and limits

This recommended practice is directed mainly toward the use of shrinkage-compensating concrete in structures, precast concrete products, slabs, both on grade and structural, and pavements. Recommendations are included for proportioning, mixing, placing, finishing, curing, and testing based on data presented in the committee's previous report and on the experience of producers, consultants, and contractors.

The recommendations herein are not applicable to self-stressing expansive cement concretes which are proportioned to produce a prestressed structure for load-bearing purposes. Procedures for proportioning, handling, and curing of self-stressing concretes are often radically different from shrinkage-compensating concretes. The fact that shrinkage-compensating concretes may attain certain levels of prestress before, or contain a small residual prestress after drying shrinkage is only incidental to their primary purpose in compensating for normal drying shrinkage.

1.4—Definitions

The definitions of terms used in this recommended practice pertaining to shrinkage-compensating concrete are:

1. *Expansive cement (general)*—A cement which when mixed with water forms a paste that, after setting, tends to increase in volume to a significantly greater degree than portland cement paste; used to compensate for volume decrease due to shrinkage or to induce tensile stress in reinforcement (post-tensioning).

2. *Expansive cement Type K*—A mixture of portland cement, anhydrous tetracalcium trialuminate sulfate ($C_4A_3\overline{S}$),* calcium sulfate ($CaSO_4$) and lime (CaO). The $C_4A_3\overline{S}$ is a constituent of a separately burned clinker that is interground with portland cement or alternately, it may be formed simultaneously with the portland cement clinker compounds during the burning process.

3. *Expansive cement Type M*—Interground or blended mixtures of portland cement, calcium aluminate cement, and calcium sulfate suitably proportioned.†

4. *Expansive cement Type S*—A type of portland cement containing a large computed tricalcium aluminate (C_3A) content and interground with an amount of calcium sulfate above the usual amount found in portland cement.

5. *Shrinkage-compensating cement*—An expansive cement so proportioned that when combined with suitable amounts of aggregate and water forms a shrinkage-compensating concrete or mortar.

6. *Shrinkage-compensating concrete* — An expansive cement concrete which when properly restrained by reinforcement or other means will expand an amount equal to or slightly greater than the anticipated drying shrinkage. Because of the restraint, compressive stresses will be induced in the concrete during expansion. Subsequent drying shrinkage will reduce these stresses but ideally, a residual compression will remain in the concrete thereby eliminating shrinkage cracking.

7. *Ettringite* ($3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O$) — The phase formed during the hydration of expansive cements which is the source of the expansive force. It is comparable to the natural mineral of the same name. This high sulfate, calcium sulfoaluminate is also formed by sulfate attack on mortar and concrete and was designated as "cement bacillus" in older literature.

Further explanation and definitions can be obtained by reference to the previous ACI Committee 223 report.[1]

1.5—General considerations

The same basic materials and methods necessary to produce high quality portland cement concrete are required to produce satisfactory results in the use of shrinkage-compensating concrete. Since the performance of the cement in minimizing cracking in concrete depends in large measure on early expansion, additional care and control must be exercised during the mixing, placing, and early curing. In some instances special procedures are necessary to insure adequate hydration at the proper time. Similarly, the structural design must be such as to insure adequate expansion to offset subsequent drying shrinkage. Details of the essential requirements necessary for successful application are dealt with in the following chapters.

The physical characteristics of the cured shrinkage-compensating concrete are usually of the same order as for other types of concrete. The durability of shrinkage-compensating concretes should be judged on the same basis as portland cement concretes.

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*Where $C = CaO$, $A = Al_2O_3$, and $\overline{S} = SO_3$.

†The Type M expansive cement produced in the United States is not to be confused with the stressing cement (SC) produced in the Soviet Union also from portland cement, calcium aluminate cement, and gypsum. The SC product is so proportioned that quick setting, fast hardening, and high early strength are obtained and, therefore, is not used in conventional concrete.

CHAPTER 2—MATERIALS

2.1—Shrinkage-compensating cements

2.1.1 *Types* — The three different shrinkage-compensating cements described in ASTM C 845[19] are designated as Type K, Type S, and Type M. The expansion of each of these cements when mixed with sufficient water is due principally to the formation of ettringite.

2.1.2 *Composition*—Approximately 90 percent of shrinkage-compensating cements consist of the constituents of conventional portland cement, with added sources of aluminate and calcium sulfate. For this reason, the oxide analysis on mill test reports does not differ substantially from portland cements described in ASTM C 150 except for the larger amounts of sulfate (typically 4 to 7 percent total SO_3) and usually, but not always, a higher percentage of aluminate (typically 5 to 9 percent total Al_2O_3). The free lime (CaO) content may also be somewhat higher.

The three types of expansive cements differ from each other in the form of the aluminate compounds from which the expansive ettringite is developed, as shown in Table 2.1.2. The reactive aluminate needed for the formation of ettringite in Type K cement is $C_4A_3\bar{S}$, in Type M cement is CA and $C_{12}A_7$, and in Type S cement is C_3A.

The kind of aluminate used influences the rate and amount of ettringite formation at early ages and thus the expansion. Total potential expansion is governed by the amount of aluminates and calcium sulfate and the rate at which they form ettringite. As with other types of portland cements, the compressive strength is principally due to the hydration of the calcium silicates.

TABLE 2.1.2—TYPES OF SHRINKAGE-COMPENSATING CEMENTS AND THEIR CONSTITUENTS

Expansive cement	Principal constituents	Reactive aluminates available for ettringite formation ($C_3A \cdot 3C\bar{S} \cdot H_{32}$)
Type K	(a) Portland cement (b) Calcium sulfate (c) Portland-like cement containing $C_4A_3\bar{S}$	$C_4A_3\bar{S}$
Type M	(a) Portland cement (b) Calcium sulfate (c) Calcium-aluminate cement (CA and $C_{12}A_7$)	CA and $C_{12}A_7$
Type S	(a) Portland cement high in C_3A (b) Calcium sulfate	C_3A

2.1.3 *Cement proportioning*—Shrinkage-compensating cements are manufactured to produce the proper amount of expansion without adversely affecting the concrete quality and retaining the normal range of concrete shrinkage. An important requirement is the selection of material proportions so that the CaO, SO_3, and especially the Al_2O_3, become available for ettringite formation during the appropriate period after the mix water is added. Determination of these proportions is based on the results of laboratory tests such as outlined in Section 2.1.8 conducted under standard conditions similar to those used for other portland cements.

2.1.4 *Hydration process*—Two basic factors essential to the development of expansion are the appropriate amount of soluble sulfates and the availability of sufficient water for hydration. Ettringite begins to form almost immediately when the water is introduced, and its formation is accelerated by the mixing. To be effective, however, a major part of the ettringite must form after attainment of a certain degree of strength; otherwise the expansive force will dissipate in deformation of the plastic or semiplastic concrete. For this reason, mixing more than required to insure a uniform mixture is detrimental since the ettringite formed during the prolonged agitation will reduce the amount available later for expansion. Ettringite formation continues during and after hardening, with proper curing, until either the SO_3 or Al_2O_3 is exhausted.

2.1.5 *Heat of hydration*—The heat of hydration or temperature rise of a shrinkage-compensating cement depends on the characteristics and type of the portland cement portion. In general, the heat of hydration falls within the range of the particular portland cement used.

2.1.6 *Fineness*—The surface area of shrinkage-compensating cement as determined by air permeability (Blaine) is not directly comparable to the surface area of portland cements. Shrinkage-compensating cement contains significantly more calcium sulfate than portland cement. Since the calcium sulfate grinds more readily than clinker, it contributes a greater part of the total Blaine value obtained.

The specific surface of a shrinkage-compensating cement has a major influence on the expansion as well as the early strength of concrete. As the surface area increases above the optimum for a given shrinkage-compensating cement with a specific calcium sulfate content, the formation of ettringite is accelerated in the plastic concrete. Thus, less expansion will be obtained in the hardened concrete. Shrinkage-compensating cement, like portland cement, produces a higher early strength if it has a higher surface area.

2.1.7 *Handling and storage*—Shrinkage-compensating cements are affected adversely by exposure to atmospheric levels of CO_2 and moisture in a similar manner as portland cements. Additionally, such exposure can reduce the expansion potential of these cements. If there is any question as to the expansive potential because of method or length of storage and exposure, the cement should be laboratory tested before use.

2.1.8 *Testing*—The expansion characteristics of shrinkage-compensating cements are determined by measuring the length changes of restrained 2 x 2 x 10 in. or 50 x 50 x 250 mm standard sand mortar prisms having a 10 in. or 250 mm gage length per ASTM C 806.[17] These tests measure the expansive potential of the cement and should be used to assess compliance with specifications for the cement. Levels of expansion will be different when job materials are used in the concrete mix.

2.2—Aggregates

Concrete aggregates which are satisfactory for portland cement concretes can also be used for shrinkage-compensating cement concretes. Good results can be obtained with normal weight, lightweight, and high density aggregates meeting the appropriate ASTM requirements. The aggregate type used, however, has a significant influence on the expansion characteristics and drying shrinkage of shrinkage-compensating cement concretes. For example, results of laboratory tests have shown that after a year, a shrinkage-compensating cement concrete containing a river gravel retained a residual expansion of 0.03 percent, whereas concrete made with the same shrinkage-compensating cement but containing a sandstone aggregate had 0.02 percent net shrinkage.

Aggregates containing gypsum or other sulfates may increase expansions and cause delayed expansion, later disruption of the concrete, or both. Significant amounts of chlorides in aggregates, such as found in beach sands, tend to decrease expansion and increase drying shrinkage. For these reasons, it is recommended that job aggregates be used in the laboratory trial mix proportioning tests.

2.3—Water

The mix water used in shrinkage-compensating concretes should be of the same quality as used in portland cement concretes.[13] If the use of wash water or water containing sulfates or chlorides is contemplated, they should be used in trial mixes to disclose possible adverse effects on the desired expansion levels of shrinkage-compensating concretes.

2.4—Admixtures

The effect of air-entraining agents, water-reducing admixtures, retarding agents, and accelerating admixtures on the expansion of a specific type or brand of shrinkage-compensating cement may be either beneficial or detrimental. Either the manufacturer of the admixture or the cement producer should be consulted as to past experience and compatibility of a specific type or brand of admixture with the cement which is to be used. Data obtained from laboratory testing and field experience show that the performance of admixtures is greatly influenced by the composition of the cement, ambient temperature, and mixing times.

In all cases, admixtures should be tested in trial mixes with job materials and proportions under simulated ambient conditions. Such tests should evaluate the admixture's influence on expansion, water requirement, air content, consistency, rate of slump loss, bleeding, rate of hardening, strength, and drying shrinkage.

In general:

(a) Air-entraining admixtures are as effective with shrinkage-compensating concretes as with portland cement concretes in improving freeze-thaw and deicer salt durability.

(b) Water-reducing admixtures may be incompatible with shrinkage-compensating concretes due to acceleration of the ettringite reaction which usually has the effect of decreasing expansion.

(c) Calcium chloride when used as an accelerator usually reduces expansion and increases shrinkage.

(d) Fly ash and other pozzolans may affect expansions and also influence strength development. They should be used with caution particularly at low temperatures.

(e) Set-retarding, water-reducing admixtures may react similar to water-reducing admixtures (see Item (b) above).

Since the methods of mixing and placing can influence admixture performance, laboratory results alone may not always correlate with job results.

Further details on the use and influence of admixtures are given in Chapter 4.

2.5—Concrete

2.5.1 *Strength*—The tensile, flexural, and compressive strength development of shrinkage-compensating concretes after expansion has been completed is similar to that of portland cement concretes under both moist and steam curing conditions.

The water requirement of some shrinkage-compensating concretes is greater than that of portland cement concrete for a given consistency. Compressive strengths, however, are at least comparable to portland cement concretes manufactured from the same clinker and having the identical cement content and aggregate proportions. As with portland cement concretes, the lower the water-cement ratio, the greater the compressive strength of shrinkage-compensating concretes.

2.5.2 *Modulus of elasticity* — The modulus of elasticity of shrinkage-compensating concretes is generally comparable to that of portland cement concretes.

2.5.3 *Volume change*—After expansion, the drying-shrinkage characteristics of a shrinkage-compensating concrete are similar to those of portland cement concrete. Also, the drying shrinkage is affected by the same factors, such as water content of the concrete mix, type of aggregate used, cement content, etc. The water content influences both the expansion during curing and subsequent contraction due to drying shrinkage. Fig. 2.5.3 illustrates the typical length change history of shrinkage-compensating and portland cement concrete bars. Shrinkage-compensating concretes of relatively high unit water content may develop some tensile stress instead of remaining in compression as shown.

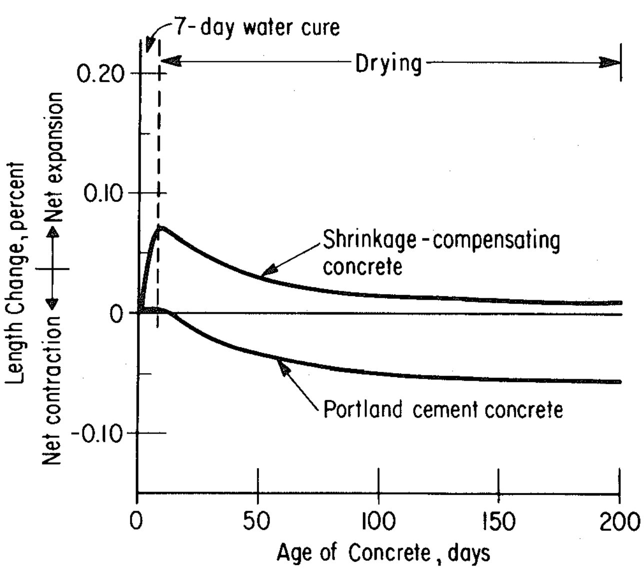


Fig. 2.5.3—Length change history of shrinkage-compensating and portland cement concretes

2.5.4 *Creep*—Data available on the creep characteristics of shrinkage-compensating concretes indicate that their creep coefficients are within the same range as those of portland cement concretes of comparable quality.

2.5.5 *Poisson's ratio* — There has been no observed difference between Poisson's ratio in shrinkage-compensating concrete and portland cement concrete.

2.5.6 *Coefficient of thermal expansion*—Tests have shown that the coefficient of thermal expansion of shrinkage-compensating concretes is consistent with that of corresponding portland cement concretes.

2.5.7 *Durability*—When properly designed and adequately cured, shrinkage-compensating concretes made with Type K, Type S, or Type M cement are comparably resistant to freezing and thawing, and deicer scaling and abrasion as portland cement concretes of the same water-cement ratio. The effect of air content and aggregates are essentially the same. Recommendations of ACI Committee 201[2] that the concrete should achieve 4000 psi (26 MPa) compressive strength prior to exposure to freeze-thaw conditions, and be at least 6 weeks old before being subjected to ice removal chemicals, should be followed.

The type of shrinkage-compensating cement and particularly the composition of the portland cement portion can have a significant effect on the durability of the concrete to sulfate exposure. Shrinkage-compensating cement made with a Type I portland cement may be undersulfated with respect to the aluminate available and therefore susceptible to further expansion and possible disruption after hardening when exposed to an external source of additional sulfates. On the other hand, shrinkage-compensating cements made with Type II or Type V portland cement clinker, and adequately sulfated, produce concretes having sulfate resistance equal to or greater than portland cement made of the same type.[14]

2.5.8 *Permeability*—The restrained expansion of shrinkage-compensating concrete produces a dense matrix which serves to reduce the permeability as compared to corresponding concrete made with other types of portland cement at the same cement content.

2.5.9 *Testing*—Compressive, flexural, and tensile strengths of shrinkage-compensating concrete should be determined in the same manner and using the same ASTM methods as for hardened portland cement concretes.

It is recommended that the amount of expansion, which is as important as strength in the performance of shrinkage-compensating concrete, be determined by measuring the length change of restrained 3 x 3 x 10 in. (76 x 76 x 254 mm) concrete prisms. This method has been used extensively in trial mixes. Details of procedures and equipment are given in "Proposed Method of Test for Restrained Expansion of Shrinkage-Compensating Concrete"[18] published by ASTM. While several other methods have been used, particularly to determine field expansions, they should be correlated with expansions determined by this method in the laboratory at the age when measured.

CHAPTER 3—STRUCTURAL DESIGN CONSIDERATIONS

3.1—General

The design of reinforced concrete structural elements using shrinkage-compensating concrete should conform to the requirements of applicable ACI standards. At the same time, adequate expansion must be provided to compensate for subsequent drying shrinkage to minimize cracking. Since the final net result of expansion and shrinkage is essentially zero, no structural consideration need normally be given to the stresses developed during this process. Provision for dead and live loads required by building codes and specifications will result in at least the same structural integrity with shrinkage-compensating concretes as with portland cement concretes.

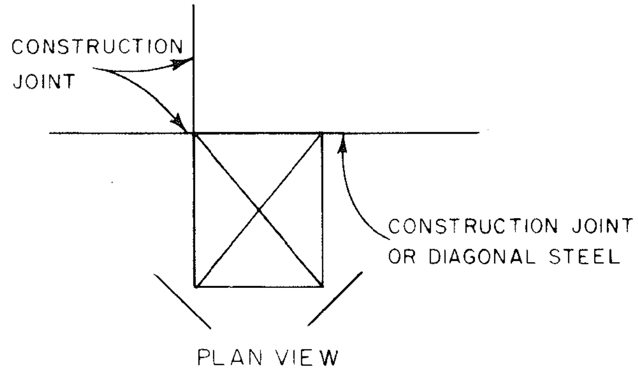
3.2—Restraint

3.2.1 *Types of restraint*—A resilient type of restraint, such as that provided by internal reinforcement, is necessary to develop shrinkage compensation. Other types of restraint, such as adjacent structural elements or subgrade friction, are largely indeterminate and may provide either too much or too little. Wherever possible, the design should, therefore, specify reinforcement as recommended in Section 3.3.

3.2.2 *Minimum reinforcement*—Established engineering design practices for structural elements normally provide a sufficient amount of steel. In some non-load-bearing members, slabs on grade and lightly-reinforced structural members, the usual amount of steel may be less than the minimum amount necessary for shrinkage-compensating concretes. For such designs, a minimum ratio of reinforcement area to gross concrete area of 0.0015 should be used in each direction that shrinkage compensation is desired. This minimum is approximately that recommended by ACI 318[3] for temperature and shrinkage stresses. At reentrant corners, at least one #4 bar approximately 3 ft (1 m) long intersecting the diagonal from the corner should be used in addition to the design steel to minimize cracking (see Fig. 3.2.2).

3.2.3 *Reinforcing location*—Although experience has shown that warping is not generally a problem, reinforcement should not be overly concentrated in one portion of any section. On the other hand, in slabs on grade where most of the drying occurs in the top portion, the reinforcement should be placed in the upper half of the slab (preferably one-third the distance from the top), while still allowing for adequate cover.

3.2.4 *Reinforcing steel*—Reinforcement can be either welded wire fabric or deformed bars meeting the requirements of ACI 318.[3] The use of plain bar reinforcement is not recommended since adequate bond may not be developed. To insure accurate positioning, deformed bars placed on chairs or tied to other fixed rods or portions of the structure are preferred. Where wire fabric is used in lieu of deformed bars, it should be in flat sheets or mats rather than rolls. Best results with fabric are obtained when the concrete is placed in two plastic layers with the fabric sandwiched in between. Hooking or pulling the fabric off the form or subgrade, or working it in from the top, should not be permitted.



(1) Eliminate corner if possible by locating construction joint. (2) If construction joint is not feasible, reinforce corner with diagonal steel to keep re-entrant corner crack from opening excessively.

Fig. 3.2.2—Re-entrant corners (pits, trenches, floor layout, truck dock, etc.)

3.3—Structural design procedures

3.3.1 *Structural design*—To provide proper safety factors, the design should be based on the strength design provisions of ACI 318.[3] This procedure will avoid the necessity of consideration of the amount of stress in the reinforcement caused by the expansion of the concrete since in the ultimate load analysis, the previous state of prestress does not influence the capacity of the section. In structural members, however, where it is anticipated that there will be high concrete expansion combined with loading at an early age, it is desirable to check that the net steel stresses caused by the expansion and loading conditions do not exceed permissible values.

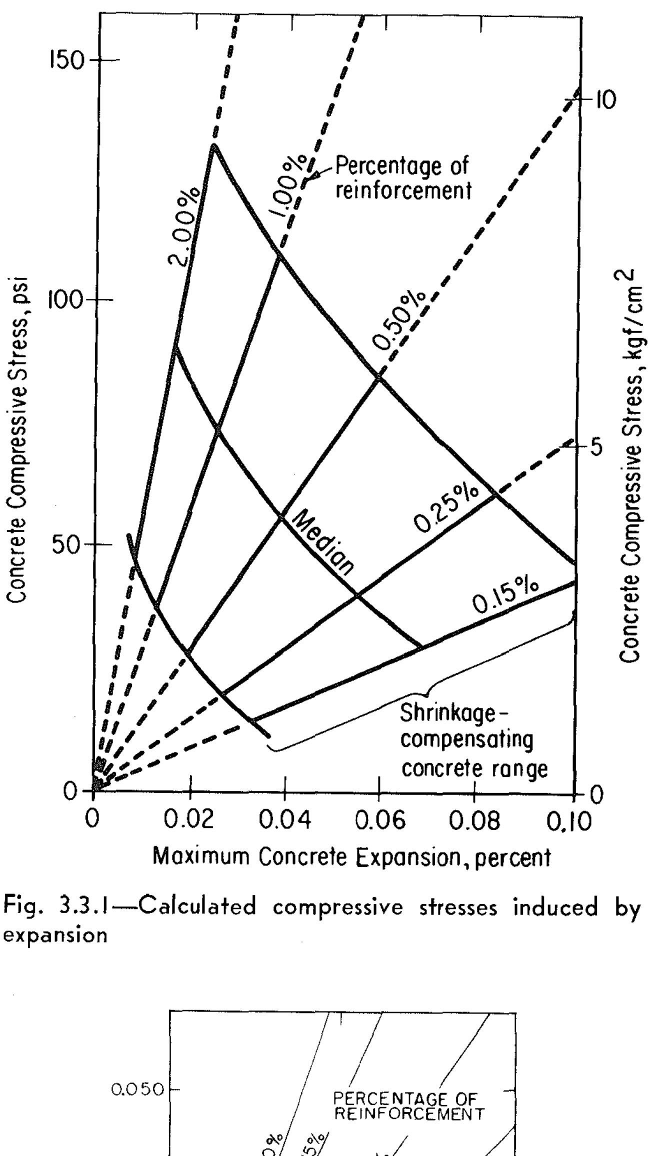


Fig. 3.3.1—Calculated compressive stresses induced by expansion

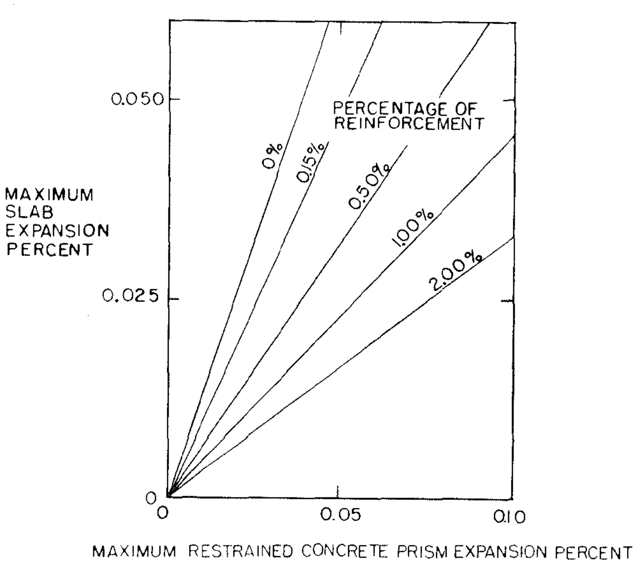


Fig. 3.3.2—Prediction of slab expansion from prism data

The magnitude of concrete stresses induced by tension in the reinforcement may be determined as follows:

Consider a reinforced concrete member which expands an amount ε_c.

If the areas of concrete and steel are A_c and A_s, respectively, then

Tensile force in steel $= \varepsilon_c E_s A_s$

Compressive force in concrete $= \varepsilon_c E_s A_s$

$$\text{Stress in concrete} = \varepsilon_c \frac{E_s A_s}{A_c} = \varepsilon_c \rho E_s$$

where $\rho = A_s/A_c$

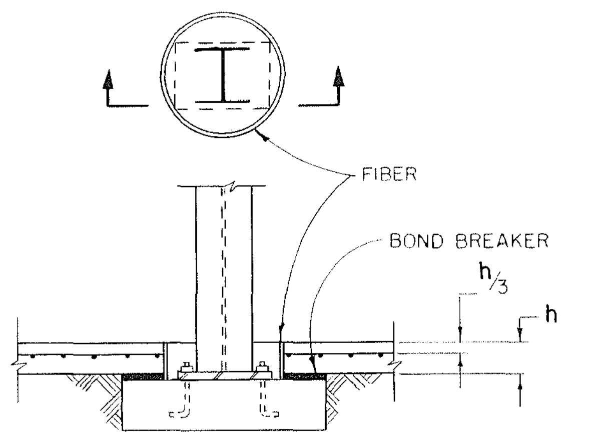
This relationship is shown graphically in Fig. 3.3.1 where E_s is taken as 29×10^6 psi (200 GPa).

As an example, a concrete member containing 0.15 percent steel which expands 0.10 percent has an induced compressive stress of 43.5 psi (300 KPa), whereas a member which only expands 0.02 percent but contains 2 percent steel has a compressive stress of 116 psi (800 KPa) (providing the expansive potential of the concrete is not exceeded).

Hence, the induced compressive stress is a function of the amount of reinforcement as well as the expansion of the concrete. As the amount of reinforcement in a member increases, the compressive stress developed in the concrete by a given expansive cement also increases. The expansive strains of a highly reinforced member are usually low, requiring only a small amount of shrinkage for the member to return to its original length and then to develop negative strains and concrete tension. The range and median of expansions generally obtained with shrinkage-compensating concretes are shown in Fig. 3.3.1.

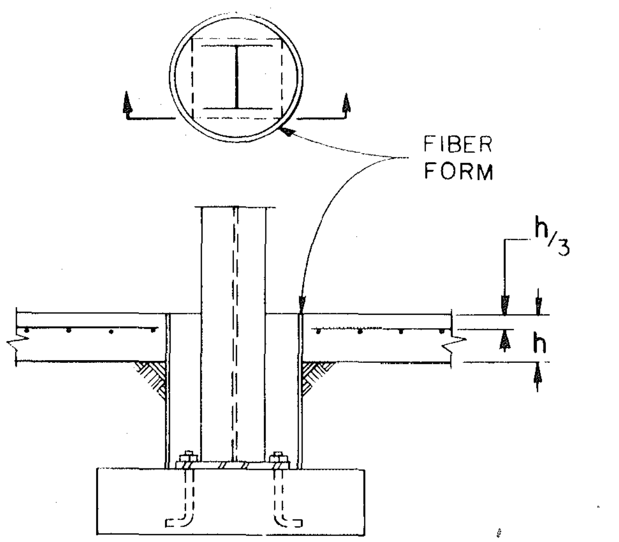
3.3.2 *Predictions of expansions*—When structural design considerations result in a reinforcement ratio greater than the recommended minimum, the level of expansion in structural slabs may be estimated from Fig. 3.3.2. This graph shows the relationship between slab and prism expansion when both are made from the same concrete and are mixed and cured under identical conditions. The prism expansion test is defined in "Proposed Method of Test for Restrained Expansion of Shrinkage-Compensating Concrete" published by ASTM.[18] Fig. 3.3.2 may also be used to estimate the required expansion of control prisms to obtain a given expansion in a structural member. To provide satisfactory shrinkage compensation, the required expansion in the structural member is recommended to be greater than, or at least equal to, the anticipated shrinkage.

3.3.3 *Deflection*—The deflection analysis to satisfy load performance criteria is made in the same manner as for other portland cement concretes. Any residual compressive stress caused by expansion will improve the service load performance by raising the bifurcation point on the load-deflection curve. Residual compressive stress, however, should not be taken into account when calculating deflections.



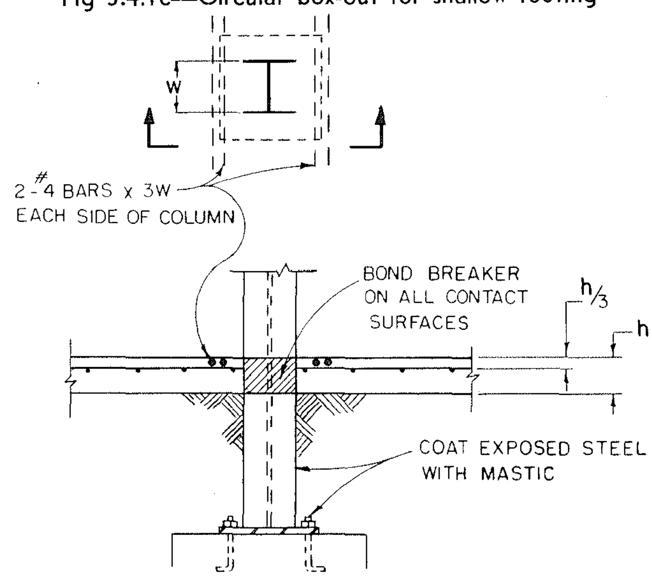
(1) Box-out must enclose base plate to prevent bonding slab to footing. (2) Bond breaker may be any material to separate slab from footing. (3) All other notes with Fig. 3.4.1a apply.

Fig 3.4.1c—Circular box-out for shallow footing



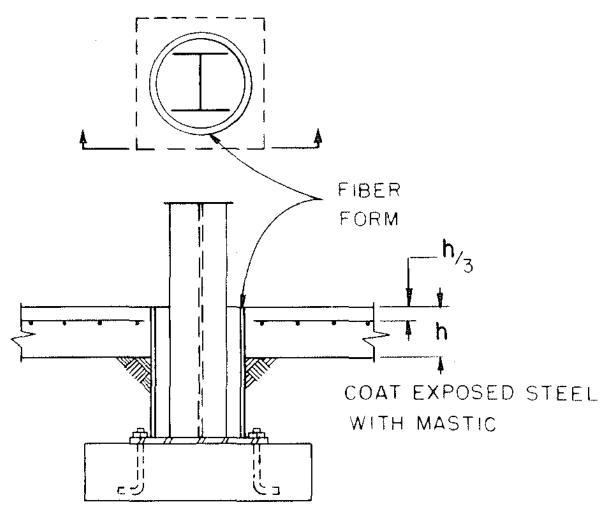
(1) For steel or concrete column. (2) Column and base plate protected. (3) Fiber form split and placed after columns are in place. (4) Concrete placed inside box-out and for slab at the same time. (5) No stress concentration point because of circular hole in the slab. (6) If smaller diameter box-out is desired, see detail in Fig. 3.4.1b.

Fig. 3.4.1a—Circular box-out for deep footing



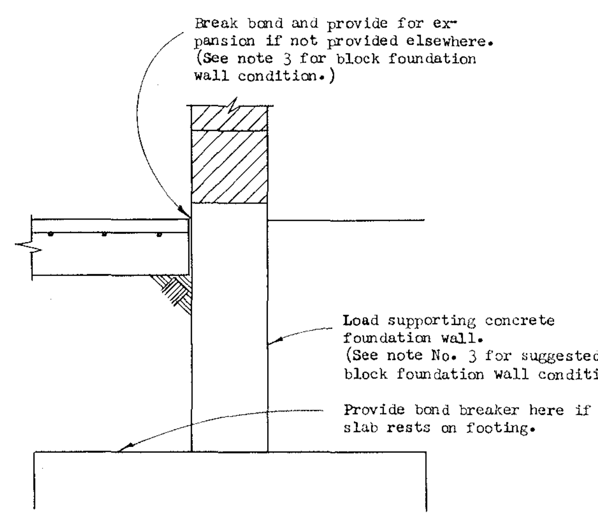
(1) No box-out provided. (2) Isolation provided by breaking bond at all concrete to steel contact surfaces with polyethylene sheet 15 lb (6.8 kg) roofing felt, heavy grease, or mastic. (3) Mastic below floor slab is for steel protection only. (4) #4 bars placed across point stress concentration areas to reduce crack tendency. (5) This detail depends on workman to place #4 bars.

Fig. 3.4.1d—Wrapped column with stress bars



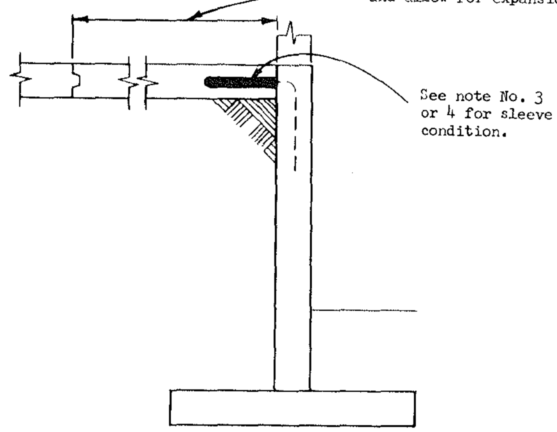
(1) Provide for smaller box-out than Fig. 3.4.1a. (2) Same notes for Fig. 3.4.1a apply.

Fig. 3.4.1b—Circular box-out for deep footing



(1) No reinforcing steel connecting slab to wall. (2) Provide waterstop if joint is to be watertight. (3) Provide 5/8 in. (15 mm) asphaltic felt for thermal movement relief at inside face of wall with block foundation wall.

Fig. 3.4.1e—Slab perimeter not tied to wall



(1) Provide for possible slab movement at construction joint rather than cracking slab because of perimeter restraint. (2) Place slab at sufficiently later date after exterior wall to allow for shrinkage relief. (3) Provide temperature relief joints in interior slab perpendicular to wall. (4) Wrap dowels to provide sleeve for horizontal movement between wall and slab if provisions in note No. 3 are not feasible. (5) Provide bond breaker between slab and wall.

Fig. 3.4.1f—Slab perimeter tied to wall

3.4—Joints

3.4.1 *Isolation joints*—Isolation joints to accommodate differential horizontal and vertical movements should be provided at junctions with walls, columns, machine foundations, and footings or other points of restraint such as drain pipes, fireplaces, sumps, stairways, etc. In addition to the normal movements, the joints may be used to accommodate the movements caused by initial expansion of the concrete. Details of isolation joints are shown in Fig. 3.4.1a through 3.4.1f.

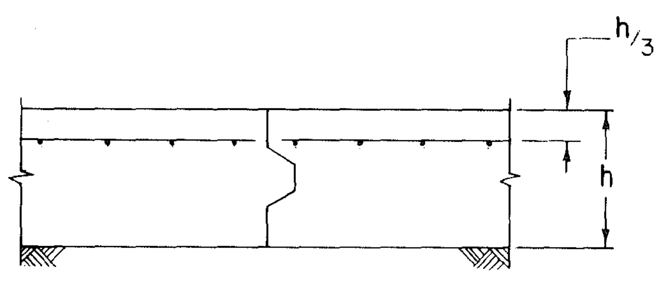
Rigid exterior restraint is not recommended since it prevents expansion of the concrete and the least amount of subsequent shrinkage will result in negative strains and concrete tension. In addition large forces will be imposed on the restraining members. In laboratory tests rigid restraints have resulted in stresses as high as 170 psi (1180 KPa). Stresses of this magnitude could produce sufficient force to damage the restraining structure. Footings, pits, walls, drains, and similar items which are so located to provide restraint should, therefore, be protected by isolating joints to prevent damage during the expansion stage of shrinkage-compensating concrete and allow the necessary expansive strain to develop. The use of compressible filler strips or joint materials is recommended for this purpose.

Column box-outs may be reduced or eliminated. A bond breaker wrapped around the column or cardboard forms brought to floor level have been satisfactory in permitting vertical movement. At the same time, the reinforcement or mesh should be increased locally in the column area where high stresses are likely to develop.

3.4.2 *Construction joints*—With the use of shrinkage-compensating concretes, checkerboard patterns spaced approximately 20 to 30 ft (6 to 9 m) when used in slab placements can be enlarged. Slabs located inside enclosed structures or where temperature changes are small, may be placed in areas as large as 16,000 sq ft (1500 m^2) without joints. For areas where temperature changes are larger or where slabs are not under enclosed structures, slab placements are normally reduced to 7000 to 12,000 sq ft (650 to 1100 m^2). The area that a work crew can adequately place and finish in a day often establishes construction joint spacing.

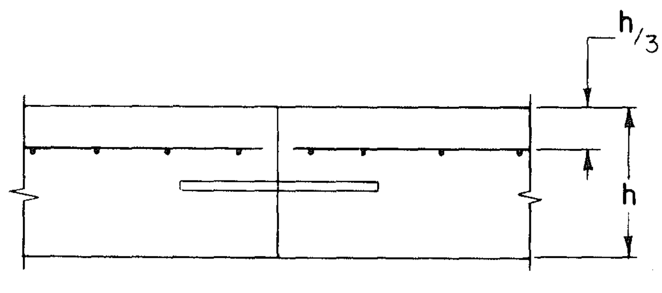
Building slab sections should be placed as square as possible. The length-to-width ratio should not exceed 3:1. For pavements, which are thicker and more heavily reinforced than building slabs, successful installations have been made with higher length-to-width ratios. In such installations, joints should be designed for the anticipated expansion. Examples of construction joint details are shown in Fig. 3.2.2 and 3.4.2a through 3.4.2e.

3.4.3 *Contraction (control) joints*—These joints are sawed, formed, or otherwise placed in slabs between construction and expansion joints. Their primary purpose is to induce controlled drying shrinkage cracking along the weakened planes



(1) Top edge at joint finished flush (no tooled edge). (2) Steel does not pass through joint. (3) Joint must be free to open due to assumed temperature contraction of slab. (4) Leave sufficient space in joint for expansion if not provided elsewhere.

Fig. 3.4.2a—Keyed construction joint



(1) Notes for Fig. 3.4.2a apply. (2) Smooth dowels are greased or wrapped to prevent bond. (3) Dowels must be placed and maintained parallel to top surface and perpendicular to joint surface. (4) Leave sufficient space in joint for expansion if not provided elsewhere.

Fig. 3.4.2b—Doweled construction joint

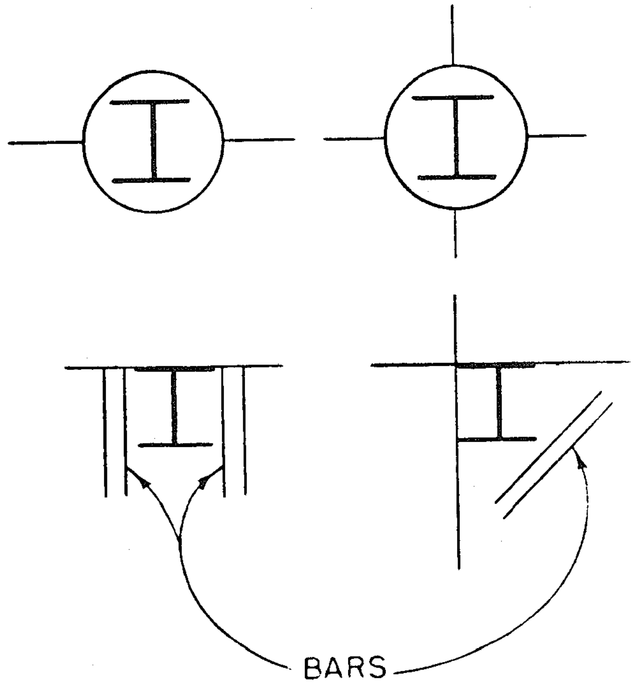
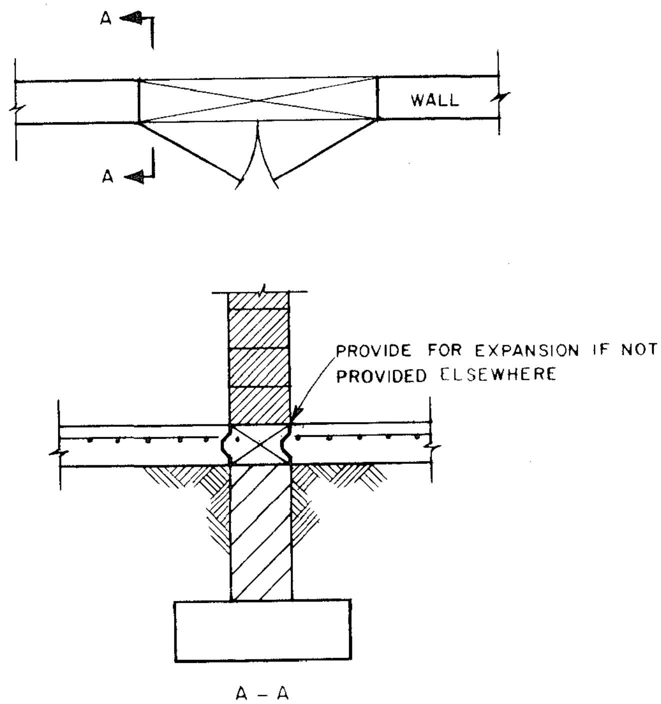


Fig. 3.4.2c—Construction joints at columns



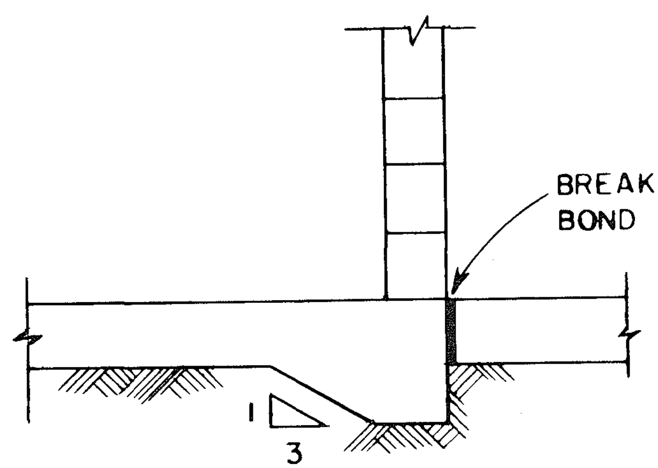
(1) Construction joints provided at door opening to allow possible slab movement parallel to wall in horizontal direction. (2) Break bond at construction joints and along wall surface. (3) Place concrete in opening at the same time as the slab by using metal keyway.

Fig. 3.4.2d—Door opening

(joints). With shrinkage-compensating concretes larger distances may be used between contraction joints. For exposed areas a maximum spacing of 100 ft (30.5 m) between joints is recommended. Where the area is protected from extreme fluctuations in temperature and moisture, joint spacings of 150 to 200 ft (45.7 to 61 m) in both directions may be used, depending on the variations anticipated. Contraction joints may be made in the same way as for portland cement concretes.

3.4.4 *Expansion joints*—The location and design of expansion joints for control of thermal movements are not changed with the use of shrinkage-compensating concrete, except that joints for thermal movements should insure that adequate expansion can take place during the expansion phase of the shrinkage-compensating concrete. In the event of high load transfer, dowel bars should be provided as shown in Fig. 3.4.2d.

3.4.5 *Details* — Suggested details of isolation joints, construction joints, door openings and wall footings for use with shrinkage-compensating concrete are shown in Fig. 3.4.1a-3.4.1f and 3.4.2a-3.4.2e. Additional details using the same basic principles should be developed by the designer as required.



(1) Thickened edges or thickened sections restrain slab movement due to temperature changes. (2) Slab on one side of wall free to move independently. (3) Construction joint at opposite end of restrained slab free to accommodate temperature movement.

Fig. 3.4.2e—Integral footing for partition walls

3.5—Connections

Connections between prefabricated shrinkage-compensating concrete members or cast-in-place members are designed in the same manner as for portland cement concretes. The design should be checked to insure that the expansive force does not produce any undesirable movement.

3.6—Formwork

Although the major proportion of the shrinkage-compensating concrete expansion takes place while it is still in the forms, no additional strengthening of the formwork has been found necessary with properly reinforced members or slabs. Generally, formwork is sufficently flexible to accommodate the expansion of the concrete.

CHAPTER 4—CONCRETE MIX PROPORTIONING

4.1—Concrete proportions

4.1.1 *Aggregates*—Selection of fine and coarse aggregate with regard to quality and proportions should follow accepted procedures used with portland cement concrete. Sections 5.3.6, 5.3.7, 5.3.8, and the appendix of ACI 211.1[4] are recommended for selecting aggregate proportions for normal weight or heavyweight concretes. Fine and coarse aggregates from a known source which have performed satisfactorily in concrete may be used in proportions previously established.

For lightweight concrete, the aggregate producer should provide information on the most effective proportion of fine and coarse aggregates and the total uncombined volumes (dry-loose basis) required to produce a cubic yard of concrete. If information from this source is not available, Section 3.2 of ACI 211.2,[5] and ACI 213[6] are recommended guides.

Additional information on proportioning aggregates for normal and lightweight concrete may also be obtained from the PCA publication "Design and Control of Concrete Mixtures."[13]

4.1.2 *Cement content*—As with portland cements, the selection of an appropriate shrinkage-compensating cement content to meet specified concrete strength requirements should be based on tests of concrete mixtures containing the materials to be used in the construction. The required strength may then be interpolated from a water-cement ratio versus compressive strength curve. If data on past performance of the shrinkage-compensating cement are not available, Table 4.1.2 may be used as a guide for setting up a trial mix program.

TABLE 4.1.2—TRIAL MIX GUIDE FOR SHRINKAGE-COMPENSATING CEMENT CONCRETE

Compressive strength at 28 days, psi (MPa)	Absolute water-cement ratio, by weight	
	Non-air-entrained concrete	Air-entrained concrete
6000 (41.4)	0.42-0.45	—
5000 (34.5)	0.51-0.53	0.42-0.44
4000 (27.6)	0.60-0.63	0.50-0.53
3000 (20.7)	0.71-0.75	0.62-0.65

When determining the required water-cement ratio and corresponding cement content, the effect of restrained expansion should be considered. Expansion increases as the cement content is increased and decreases as the cement content is lowered. A lower limit of 515 lb (235 kg) of cement per cubic yard of concrete is recommended to achieve the required expansion with minimum reinforcement (0.15 percent). In all cases, a qualified laboratory should determine the expansion of the concrete by means of restrained 3 x 3 x 10 in. (76 x 76 x 254 mm) prism specimens as described in "Proposed Method of Test for Restrained Expansion of Shrinkage-Compensating Concrete."[18] The effect of amount of reinforcement on expansion of shrinkage-compensating concrete is discussed in Chapter 3.

When shrinkage-compensating cements are specified for structures which are designed and constructed in accordance with ACI 318,[3] the requirements of Chapter 4 "Concrete Quality," will control strength, concrete proportions, and evaluation of concrete. To properly apply the provisions of Chapter 4 of ACI 318, it is necessary to become thoroughly familiar with Chapter 4 of the Commentary to the Building Code prepared by ACI Committee 318, and follow the suggestions and instructions for initial trial batches.

If trial batches are made in accordance with Section 4.2.3 of the Code, Table 4.1.2 should be used as a guide where satisfactory performance of the shrinkage-compensating cement has not been previously established. When strength data from laboratory trial batches or field experience are not available, the lower value indicated in Table 4.1.2 should be used as the maximum permissible water-cement ratio for the concrete.

4.1.3 *Water requirement*—The design water requirement of some shrinkage-compensating cements may be as much as 10 to 15 percent more than Type I and Type II portland cements. Increased water requirements can be attributed to variations in the hydration rates which are influenced by the chemical composition of the cement and such physical properties as cement fineness, concrete temperature, and mixing procedure. The additional water combines with the expansive component at a very early age so the free water available at time of placement is approximately the same as for portland cement concretes.

Experience and test results of job concrete have shown that a properly designed mixture containing shrinkage-compensating cement with an initially higher water requirement will produce comparable strengths to the same mixture containing portland cement with a lower water content and equivalent cement contents.

The additional water requirement may be determined by the mix proportioning procedures discussed in Section 4.3. It should be noted that water added to any concrete mixture beyond that required in the initial design will result in reduced strengths.

4.2—Admixtures

4.2.1 *Air-entraining admixtures*—Air-entraining admixtures which comply with ASTM C 260[7] may be used for the same purpose with shrinkage-compensating concretes as with other types of portland cement concretes. Generally, the same amount of a given air-entraining admixture will produce a comparable percentage of entrained air, all other conditions being equal.

4.2.2 *Water-reducing, retarding, and water-reducing retarding admixtures*—Some ASTM C 494 Types A, B, and D admixtures are not compatible with certain shrinkage-compensating cements. It is recommended that such admixtures be tested before acceptance, using the particular cement and other materials selected for the job. Special attention should be given to their effect on slump, restrained expansion, and drying shrinkage since it has been found that the use of certain retarding, water-reducing, or water-reducing and retarding admixtures with some shrinkage-compensating cements have resulted in excessive slump loss, a substantial loss of expansion, and excessive drying shrinkage. Generally, admixtures which are acceptable may be used in the normal dosage recommended for Type I or Type II portland cement concretes under moderate temperature conditions. During hot weather, larger than normal dosages of acceptable ASTM C 494[8] retarding Types B and D have on occasion been used successfully to delay initial setting time of some shrinkage-compensating concretes.

4.2.3 *Accelerators*—Calcium chloride is generally not recommended for use in expansive cement concrete due to its effect in reducing expansion and increasing subsequent drying shrinkage. Some expansive cement producers, however, permit its use under certain conditions but limit the content to 1 percent by weight of cement. It is recommended that the cement producer be consulted before calcium chloride is used in shrinkage-compensating concrete. If used, calcium chloride or ASTM C 494[8] Types C and E admixtures should be added in solution in accordance with accepted practice for Types I and II cement concretes.

4.3—Consistency

Good results can be obtained using slumps at time of placement within the maximum range specified by ACI 211.1[4] for the work involved when concrete temperatures do not exceed 75 F (24 C). At higher concrete temperatures, the following maximum slumps at point of placement are recommended:

Type of construction	Slump (in. cm)
Reinforced foundation walls and footings	5 (13)
Plain footings, caissons, and substructure walls	4 (10)
Slabs, beams, reinforced walls	6 (15)
Building columns	6 (15)
Pavements	4 (10)
Heavy mass construction	4 (10)

With ready-mixed concrete operations, the delivery time of concrete between the batch plant and placement may be as short as 20 min or as long as 1½ hr. Ettringite will begin to form during this period in some shrinkage-compensating cement concretes resulting in a premature stiffening and slump loss of 2 to 3 in. (50-80 mm). It is, therefore, essential that sufficient slump within maximum allowable water limits be provided at the batch plant to insure the specified or desired slump is obtained at the job site. The importance of taking this slump loss into account in selecting proportions for these types of shrinkage-compensating cement concretes cannot be overemphasized. It becomes even more important during hot weather when concrete temperatures are relatively high and reactions are accelerated. While normal delivery time of ready-mixed portland cement concrete under adverse hot weather conditions results in a significant slump loss, some shrinkage-compensating cement concretes develop an even greater slump loss under the same hot weather conditions.

Slump loss controls in hot weather which are successful for portland cement concretes are equally effective for shrinkage-compensating cement concretes. Stricter enforcement is recommended, however, when expansive cements are used because of the possible greater slump loss. Recommended controls include cooling the concrete, reducing the speed of the truck mixer drum to a minimum during travel and waiting time at the job site and efficient truck scheduling so as to reduce the period between mixing and delivery to an absolute minimum. When job locations require extended travel time, dry batched truck delivery with job site mixing is effective so long as the cement is charged on top of the aggregates without turning the drum.

For a more complete discussion of hot weather concreting, reference should be made to ACI 305. Attention is particularly directed to Chapters 1, 2, and 3 which deal with hot weather control of concretes properties, production, and delivery. The objectives are to identify hot weather problems and recommend concreting practices which will alleviate adverse effects likely to be experienced.

4.4—Mix proportioning procedures

Trial mixes using job materials should be made in the laboratory at the approximate concrete temperatures anticipated in the field. The following procedures have been successful in developing satisfactory batching plant and job control programs under differing conditions:

4.4.1—When the time between addition of mix water and placement is not more than 15 min such as precast or job-site mixing, the total mixing water required will be comparable to that of a Type I or Type II portland cement concrete for the specified slump. Trial batches to develop satisfactory aggregate proportions, cement content, and the water requirement should follow the recommendations set forth in Section 4.1. The mixing procedure in ASTM C 192[10] should be used.

4.4.2—When the water is added at the batch plant and where delivery will require normal travel time (30-40 min) in a truck mixer whether truck or central-mixed, or when expected concrete temperature will exceed approximately 75 F (24 C), some slump loss can be expected and must be compensated for by a relatively high initial slump to produce the slump required at the job site. Under such conditions, both of the following procedures for trial batch tests have been used successfully:

Procedure A

1. Prepare the batch using ASTM C 192[10] procedures but add 10 percent additional water over that normally used for Type I cement.
2. Mix initially in accordance with ASTM C 192 (3 min mix followed by 3 min rest and 2 min remix).
3. Determine the slump and record as initial slump.
4. Continue mixing for 15 min.
5. Determine the slump and record as placement slump. Experience has shown this slump loss correlates with that expected for a 30-40 min delivery time. If this slump does not meet the required placement specification limits, discard and repeat the procedure with an appropriate water adjustment.*
6. Cast compressive strength and expansion specimens and determine the plastic properties—unit weight, air content, temperature, etc.

Procedure B

1. Prepare the batch using ASTM C 192[10] procedures for the specified slump.
2. Mix in accordance with ASTM C 192[10] (3 min mix, 3 min rest and 2 min remix) and confirm the slump.
3. Stop the mixer and cover the batch with wet burlap for 20 min.
4. Remix 2 min adding water to produce the specified placement slump. The total water (initial plus the remix water), is that required at the batching plant to give the proper job-site slump after a 30-40 min delivery time.
5. Cast strength and expansion specimens and determine the plastic properties—unit weight, air content, temperature, etc.

4.4.3—Whenever possible, trial mixes should be made to insure satisfactory and economical results. If a trial batch is not made, either of the following approximations have often given satisfactory results when time and concrete temperature conditions are the same as in Section 4.4.2:

(a) Add approximately 10 percent to the water requirement for the same mix as if a Type I or Type II cement were used and proportioned under ACI 211 standards.[4,5]

(b) Use a water-reducing admixture recommended by the cement manufacturer known to be compatible with the shrinkage-compensating cement used, and maintain the same amount of mix water as if no admixture were used.

CHAPTER 5—PLACING, FINISHING, AND CURING

5.1—Placement

The plastic characteristics of all three types of shrinkage-compensating concrete are sufficiently similar to concretes made with Types I or II portland cement so that no special equipment or techniques are required for satisfactory placement. The recommendations set forth in ACI 304[11] should be followed where applicable. Successful placements have been made by wheelbarrow, mixer truck, bucket, conveyor, and shotcrete. In general, shrinkage-compensating concretes have more cohesiveness or "fat" than portland cement concretes and less tendency to segregate. For this reason, they are especially adaptable to pumping and a large percentage of shrinkage-compensating concrete has been placed by that method. Shrinkage-

*Drum-type laboratory mixes operating at normal speed when used for extended mixing slump loss tests may give a slump loss value approximately 20 percent less than determined by counterflow pan-type mixers. The pan-type mixer will more closely approximate the loss to be expected for the 30-40 min travel in a truck mixer.

compensating concretes have also been used without difficulty in the manufacture of pipe in precast operations and in paving machines.

The same placing recommendations as for portland cement concretes are equally important for shrinkage-compensating concrete. In addition, however, the characteristics of shrinkage-compensating concrete require that certain precautions be followed to insure adequate expansions and satisfactory results.

1. Where the plastic concrete will be in contact with an absorptive material such as dry soil or previously placed dry concrete, the base or subgrade should be thoroughly wetted. Sprinkling lightly is not sufficient. Recommended practice is to soak the base the evening before placement and sprinkle ahead of the placement as necessary. It is also good practice to wet the forms and reinforcement for structural concrete, particularly in hot weather.

2. In hot, dry, and windy placing conditions, all concretes tend to lose moisture unevenly and may develop plastic shrinkage cracks. Experience has shown that with shrinkage-compensating concrete, plastic shrinkage cracking is more prevalent because of water required for the early formation of ettringite. Finishing difficulties may be increased because of nonuniform moisture loss between top and bottom surfaces during the drying period, particularly when the concrete is placed directly over a vapor barrier. Where a vapor barrier is required, it is recommended that it be covered with a minimum of 3 in. of sand, thoroughly wetted, before placing shrinkage-compensating concrete. This practice results in more even moisture loss of the shrinkage-compensating concrete, less plastic shrinkage cracking, and protection of the vapor barrier during placement.

3. Care must be taken to maintain the reinforcement in its proper position during placement and consolidation to assure that it provides the required restraint. At the same time, the concrete should be properly consolidated to insure good bond with the steel.

4. Special precautions should be taken to avoid placing delays at the job site when using ready mixed concrete. A substantial increase in mixing time over that assumed when selecting mix proportions increases the slump loss and any water added to maintain consistency not only decreases the strength but may also reduce the expansion to unacceptable levels.

5. Concrete temperature and time in the mixer (from intermingling of cement and damp aggregate) are important factors because of their effect on expansion. It is recommended that the temperature of the shrinkage-compensating concrete at the time of placement not exceed 90 F (35 C) and the mixing time for shrinkage-compensating concrete at temperatures above 85 F (30 C) be limited to 1 hr. For shrinkage-compensating concrete below 85 F (30 C), the mixing time should be a maximum of 1½ hr.

5.2—Finishing

The cohesiveness or "fat" inherent in expansive cements provides excellent finishing qualities. Its behavior is similar to air-entrained concrete including the same stickiness, but this usually presents no problems. Similarly, there is little or no bleeding even though a relatively high slump may be used. Due to lack of bleed water, however, there is a tendency for finishers to start too soon. On the other hand, in warm weather, shrinkage-compensating concrete will typically set faster than Type I or Type II portland cement concretes and finishing may start somewhat sooner than normal. For these reasons, finishing may require greater manpower for a shorter period than would be typical for the usual concrete finishing operations under similar conditions.

In general, satisfactory results will be obtained when the recommendations of ACI 304,[11] Section 10.3, are followed, together with the more detailed recommendations of ACI 302, Chapter 7.

5.3—Curing

Shrinkage-compensating concrete, as with all portland cement concrete, requires continuous curing at moderate temperatures for several days after final finishing operations to prevent early drying shrinkage and to develop strength, durability, and other desired properties. Any deficiencies in the method of curing may also reduce the amount of initial expansion which is needed to offset later drying shrinkage. The usually accepted methods of curing are satisfactory for shrinkage-compensating concrete; however, those that provide additional moisture to the concrete such as ponding, continuous sprinkling, and wet coverings are preferred to insure adequate water for ettringite formation and expansion. Other methods such as moisture-proof covers and sprayed-on membranes have been successfully utilized, provided that coverage is complete so that it prevents loss of moisture from the entire concrete surface. Curing of shrinkage-compensating concrete should be continued for a minimum of 7 days.

Curing of concrete flatwork should commence immediately after final finishing. It may be necessary to fog spray or cover the surface of the concrete temporarily if other methods of curing are delayed, especially in hot, dry, or windy weather.

If a liquid curing membrane is used, it is recommended that it be applied in two directions, at a coverage rate suggested by the manufacturer, immediately following the final finishing as it progresses. To accomplish this, power spray equipment capable of covering large areas more rapidly should be used rather than small, portable spray tanks.

For architectural or structural concrete, the normally accepted practice of curing with the formwork in place is adequate for shrinkage-compensating concrete. All uncovered surfaces should receive additional curing by one of the accepted methods. In hot weather, soaker hoses or water sprays should be used to supplement the protection of the in-place formwork. If the forms must be removed prior to 7 days, one of the other accepted methods of curing should then be employed for the balance of the curing period.

Shrinkage-compensating concrete should be protected during the initial curing period against extremes of temperatures during either cold or hot weather periods. The methods recommended are those described in ACI 305[9] and ACI 306.[12]

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Steel Reinforcement Properties and Availability

Reported by ACI Committee 439

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The material properties of the various types of steel reinforcement produced for use in the United States are described. Deformed reinforcing bars, plain bars and wire, welded wire fabric, bar mats, and prestressing reinforcement are the reinforcement types examined. The requirements and restrictions of the pertinent ASTM specifications are reviewed. Included is a discussion of the test requirements of deformed reinforcing bars. The availability of the various types and sizes of reinforcement in the United States is also summarized.

Keywords: bend tests; bending (reinforcing steels); deformed reinforcement; ductility; **mechanical properties; prestressing steels;** reinforced concrete; **reinforcing steels;** specifications; spiral reinforcement; tensile strength; welded wire fabric; yield strength.

CONTENTS

*Members of the subcommittee which prepared the report.

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GENERAL

Requirements for deformed reinforcement are stated in Section 2.1 of "Building Code Requirements for Reinforced Concrete (ACI 318-71)" (1976 Supplement) as: "Deformed reinforcing bars, bar and rod mats, deformed wire, welded plain wire fabric, and welded deformed wire fabric conforming to Sections 3.5.1, 3.5.4, 3.5.7, 3.5.6, or 3.5.8, respectively."

Requirements for reinforcement are stated in Section 3.5 of ACI 318-71 (1976 Supplement) as: "Reinforcement shall be deformed reinforcement, except that plain reinforcement may be used for spirals or tendons, and reinforcement consisting of structural steel, steel pipe, or steel tubing may be used as specified in this Code."

In other words, steel reinforcement is all reinforcing bars, spirals, bar mats, welded wire fabric, and prestressing steel. The purpose of this report is to describe these types of reinforcing, excluding wire fibers, glass fibers, expanded metal, and the like, in terms of material properties and availability. With respect to reinforcing bars the description will include significant items of testing and the current exceptions placed by ACI on the ASTM specifications.

REINFORCING BARS

Material properties

Most of the properties of reinforcing bars of interest to the designer are defined by ASTM. However, ACI 318-71 defines certain mechanical properties somewhat differently from corresponding ASTM definitions. Furthermore, the significance of certain properties as defined by ASTM is not readily apparent, and some special properties are not defined by ASTM. Therefore, a brief review of the design significance of various mechanical properties is appropriate.

Strength

Of prime importance in design is the yield strength (for nonprestressed reinforcement) and tensile strength (for prestressed reinforcement) (see Table 1 for a summary of material strengths). For nonprestressed reinforcement, ACI limits the yield strength over 60 ksi (414 MPa) to a stress corresponding to a maximum (EUL*) strain of 0.35 percent. The purpose is to provide a basis for standard structural computations in accordance

*Extension under load.

TABLE 1—SUMMARY OF MATERIAL STRENGTHS

Product	ASTM Specification	Grade	Yield strength minimum, psi*	Tensile strength minimum, psi*
Reinforcing bars	A 615	40 60	40,000 60,000	70,000 90,000
	A 616	50 60	50,000 60,000	80,000 90,000
	A 617	40 60	40,000 60,000	70,000 90,000
	A 706	60	60,000 (78,000 maximum)	80,000
Bar mats	A 184	Same as reinforcing bars		
	A 704	Same as reinforcing bars		
Wire—Smooth	A 82		70,000	80,000
Deformed	A 496		75,000	85,000
Welded wire fabric—Smooth	A 185		65,000	75,000
Deformed	A 497		70,000	80,000
Prestressed bar	A 722	Type I	127,500	150,000
		Type II	120,000	150,000
Prestress wire	A 421		188,000-200,000	235,000-250,000
Prestress strand	A 416	250 270	212,500 229,500	250,000 270,000

*10,000 psi = 68.9 MPa

TABLE 2—MECHANICAL PROPERTIES OF REINFORCING BARS

Type of steel and ASTM designation/year	Size range	Grade	Yield, psi (MPa)*	Tensile strength, psi (MPa)	Minimum percentage elongation in 8 in. (203 mm)	Bend pin diameter§ (d = nominal bar diameter)
Billet, A 615-76a	3-11	40	40,000 (276)	70,000 (483)	#3, #7 ... 11 #4, #5, #6 ... 12 #8 ... 10 #9 ... 9 #10 ... 8 #11 ... 7	#3, #4, #5 ... 4d #6 and larger ... 5d
	3-11, 14, 18	60	60,000 (414)	90,000 (621)	#3. #4, #5, #6 ... 9 #7, #8 ... 8 #9, #10, #11, #14, #18 ... 7	#3, #4, #5 ... 4d #6 ... 5d #7, #8 ... 6d #9, #10, #11 ... 8d #14, #18† .. 10d(90 deg)
Rail, A 616-76	3-11	50	50,000 (345)	80,000 (552)	#3, #7 ... 6 #4, #5, #6 ... 7 #8, #9, #10, #11 ... 5	#3 through #8 ... 6d #9, #10 ... 8d #11 ... 8d (90 deg)
	3-11	60	60,000 (414)	90,000 (621)	#3, #4, #5, #6 ... 6 #7 ... 5 #8, #9, #10, #11 ... 4½	#3 through #8 ... 6d #9, #10 ... 8d #11 ... 8d (90 deg)
Axle, A 617-76	3-11	40	40,000 (276)	70,000 (483)	#3, #7 ... 11 #4, #5, #6 ... 12 #8 ... 10 #9 ... 9 #10 ... 8 #11 ... 7	#3, #4, #5 ... 4d #6 and larger ... 5d
	3-11	60	60,000 (414)	90,000 (621)	#3, #4, #5, #6, #7 .. 8 #8, #9, #10, #11 ... 7	#3, #4, #5 ... 4d #6 ... 5d #7, #8 ... 6d #9, #10, #11 ... 8d
Low alloy, A 706-76	3-11, 14, 18	60	60,000 minimum (414) 78,000 maximum (538)	80,000‡ (552)	#3, #4, #5, #6, #7, #8, #9, #10, ... 14 #11 ... 12 #14, #18 ... 10	#3, #4, #5 ... 3d #6, #7, #8 ... 4d #9, #10, #11 ... 6d #14, #18 ... 8d

*Yield point or yield strength. See specifications.
†Under supplemental requirements of ASTM A 615 only.
‡Tensile strength shall not be less than 1.25 times the actual yield strength.
§Test bends 180 deg unless noted otherwise.

with generally accepted theoretical equations. However, ASTM specifications have different definitions for yield strength that result in more practical controls on production. It has been shown for reinforcing bars* that, with the ASTM controls, as-produced reinforcing steel with a specified yield strength of 60 ksi (414 MPa) or less generally exhibits a stress equal to or greater than the specified yield strength at a strain not exceeding 0.35 percent. For designs assuming yield strengths greater than 60 ksi (414 MPa), special arrangements must be made with the producer and/or fabricator to insure that the steel with the desired yield characteristics will be available. Alternatively, the commonly used design formulas could be altered to reflect (1) a yield strength corresponding to the strain defined for ASTM acceptance tests and (2) possibly the "helpful" effect of any concrete creep that occurs before the structure experiences the maximum design loadings.

Ductility

The tensile-test ductility requirements (after-failure measurement of the elongation of a tensile specimen) as specified by ASTM have resulted in reinforcing steels that have generally experienced only a very minor number of fractures in fabrication or in situ in concrete structures, even structures subject to earthquakes. However, it is important in any inelastic analysis to realize that the "useful" ductility is limited to the strain corresponding to the greatest stress on the engineering stress-strain curve, which may be less than half the ultimate ductility. Ductility is not an important parameter in members subjected primarily to compression, but can be critical in flexural members or columns with significant bending, as in a structure subjected to an earthquake, only if the percentage of reinforcing steel is very low so that there is a possibility of steel rupture before concrete crushing. Rupture of longitudinal steel has generally been precluded in bending members by using steels with ductilities as defined by ASTM

*Wiss, Janney, Elstner and Associates, "Final Report on Bar Tests for the Committee of Concrete Reinforcing Bar Producers—American Iron and Steel Institute," unpublished tests summarized in a private communication, Northbrook, Ill., Apr. 30, 1970.

TABLE 3—REINFORCING BARS—CHEMICAL RESTRICTIONS AND SPECIAL PROPERTIES

ASTM specification	Chemical restrictions	Special properties
A 615-76a	0.05 percent maximum phosphorus	Supplement S1 for bent #14 and #18 bars One tensile retest allowed when: (a) Yield value of original sample less than 1000 psi (6.9 MPa) below specification minimum, or (b) Tensile value of original sample less than 2000 psi (13.8 MPa) below specification minimum, or (c) Elongation percentage of original sample is less than 2 percent points below specification minimum.
A 616-76		Bars to be rolled from standard section Tee rails only Bar mark to include rail symbol
A 617-76		Bars to be rolled from carbon steel Axles for cars and locomotive tenders in standard journal sizes. Bar mark to include *A*
A 706-76	Check analysis variation Carbon 0.30 percent maximum + 0.03 percent Manganese 1.50 percent maximum + 0.05 percent Phosphorus 0.035 percent maximum + 0.008 percent Sulfur 0.045 percent maximum + 0.008 percent Silicon 0.50 percent maximum + 0.05 percent Carbon equivalent not to exceed 0.55 $CE = \%C + \frac{\%Mn}{6} + \frac{\%Cu}{40} + \frac{\%Ni}{20} + \frac{\%Cr}{10} - \frac{\%Mo}{50} - \frac{\%V}{10}$	One tensile retest allowed when: (a) Yield value of original sample less than 1000 psi (6.9 MPa) below specification minimum, or (b) Tensile value of original sample less than 2000 psi (13.8 MPa) below specification minimum, or (c) Elongation percentage of original sample is less than 2 percent points below specification minimum.

and by imposing lower limits on steel percentage, such as given in Section 10.5 of ACI 318-71. Ductility is obviously important in members subjected to membrane tension, as may be the case in some structural components, such as shearwalls, that may be subjected to seismic loadings.

Special properties not defined by ASTM

ASTM specifications for steel reinforcement do not include restrictions regarding fatigue or impact properties or properties at high strain rates. In practice, reinforcing steel has generally not been subject to fatigue or impact failures. No practical methods for testing impact properties of reinforcing steel have been devised; Charpy tests on machined specimens do not really reflect impact properties of a deformed bar or wire. High strain rates result in higher yield strengths and, to a lesser extent, higher tensile strengths. Ductility generally is not sensitive to strain rate.[1-4]

Although ACI 318-71 allows for the use of reinforcing steel with specified minimum yields in excess of 60,000 psi (414 MPa) up to 80,000 psi (552 MPa), there is no current ASTM standard specification for such a material. At the present time ASTM has a task group studying the possibility of a Grade 80 specification. See Reference 5 for a further discussion of Grade 80 reinforcing bars. Tables 2 and 3 summarize the mechanical and/or chemical restrictions or special properties of the ASTM specifications.

Availability

Reinforcing bars rolled to the ASTM A 615 specification are the most commonly specified and consequently available throughout the country. Most of the major producers roll reinforcing bars in such a manner as to make a 60 ft (18.3 m) length the "stock" or standard length available without special order. Lengths longer than 60 ft-0 in. (18.3 m) normally require special arrangement with the supplier. Producers may require large quantities of special or overlength material, usually heat lots, in order to roll lengths over 60 ft-0 in. (18.3 m). Stock material in 20 ft-0 in., 30 ft-0 in., and 40 ft-0 in. (6.1 m, 9.1 m, 12.2 m) lengths is normally available, usually in the smaller bar sizes (#3-#6).

Rail and axle steel (ASTM A 616 and A 617) is not generally available except in a few areas of the country. The majority of construction uses billet steel (ASTM A 615).

Reinforcing steel is generally furnished shop fabricated (sheared, bent, bundled, and tagged for identification) from an organization that contracts to furnish the reinforcing bars. The "fabricator" usually prepares the placing drawings from the structural drawings. The reinforcing bars are fabricated from the information developed on the placing drawings. Fabricators obtain the stock length material directly from a rolling mill. Fabricating shops are located throughout the country.

Reinforcing bars with special chemical or physical properties may be developed for particular applications such as nuclear power plants and anchor bolts for transmission towers. Such materials with special chemistry or properties require large quantity orders.

At the present time Grade 40 and Grade 60 material is equally available. The economies that are available by designing and using Grade 60 material, as provided in ACI 318-71, are reducing the demand for Grade 40 material. This reduction, however, is long term. There has been a noticeable reduction in the usage of some of the large size bars in Grade 40. ASTM A 615-72 did not include #14 and #18 bar sizes in Grade 40; additionally ASTM A 615-74a, A 615-75, and A 615-76a no longer include Grade 75 material in any bar sizes.

In general, ASTM A 615 Grades 40 and 60 in lengths up to 60 ft-0 in. (18.3 m) in bar sizes #3 through #11 are readily available in all parts of the country. This statement regarding availability appears to contradict Footnote *a* of Table 2 in ASTM A 615 which states that bar sizes #7 through #11 in Grade 40 may not readily be available throughout the country. This footnote was intended to reflect the possible impact of the general shortage of steel products that existed in 1974 and early 1975. Currently that general shortage situation is not present. Grade 60 #14 and #18 bars are generally available but are not sizes usually kept in a fabricator's inventory.

Since ASTM A 706 was initially issued in 1975, the demand and consequently the availability for this material has not been clearly defined. Even though the material, by specification, is defined for all bar sizes it appears that the primary application will be in structures where ductility/weldability are primary concerns, particularly in the larger bar sizes.

Welding

ASTM A 615 does not include weldability as part of the specification. ASTM A 706 does include weldability, hence the specific chemical composition requirements and calculation of the carbon equivalent.

However, if there is a necessity for welding ASTM A 615 material, the provisions of the American Welding Society "Reinforcing Steel Welding Code" (AWS D12.1-75) must be followed. The welding procedures defined in AWS D12.1-75 require the calculation of the carbon equivalent as in the ASTM A 706 specification. The carbon equivalent calculation requires that the chemistry of the bars to be welded be known either from information provided by the producer or by testing a sample.

Welding of material furnished under ASTM A 706 must also follow the procedures outlined in AWS D12.1-75, depending on the actual carbon equivalent value. Proper heat and electrodes must be used to achieve acceptable welds. The prohibition against "tack" welding is also valid when utilizing ASTM A 706 material.

Material testing

ASTM specifications require that the yield strength, tensile strength, and elongation of a representative sample of each heat of material rolled per size be determined. A bend test is also required. Depending on the results of the tests the material is considered acceptable or unacceptable. The test methods and procedures used are outlined in ASTM A 370.

ASTM A 615 reinforcing bar specifications require full section (as rolled) specimens to be tested for sizes #3-#11 Grade 40, and #3-#10 Grade 60 with the option to use reduced section tensile test specimens on Grade 60 #11, #14, and #18 bars. In the reduced section test the specimen is machined down to a diameter less than the as-rolled condition. While the reduced section tests yield more consistent results than the full section tests, full section testing is often defined in a project specification in order to have more realistic test results. Many major producers test all reinforcing bars using full sections. However, reduced section testing may be done when access to the testing equipment required to test the large diameter bars to the full tensile (ultimate) strength is not available. ASTM A 706 requires tests to be made on full section (as-rolled) specimens only. No allowance is made for reduced section testing.

Until recently the test requirements of ASTM A 615 were considered to be sufficient. However, with the advent of nuclear power plants and the associated special design and quality control/assurance requirements, additional test criteria have been established for reinforcing bars to be

TABLE 4—BEND TEST REQUIREMENTS

(180 deg bends unless noted around pins having diameters listed)

Bar size	ASTM A 615-76 Grade 40	ASTM A 615-76 Grade 60	ASTM A 616	ASTM A 617 Grade 40	ASTM A 617 Grade 60	ACI 318-71* Grade 40	ACI 318-71* Grade 60	ASTM A 706 Grade 60	AASHTO M31 Grade 40	AASHTO M31 Grade 60
3, 4, 5	4d	4d	6d	4d	4d	3½d	3½d	3d	4d	4d
6	5d	5d	6d	5d	5d	5d	5d	4d	4d	5d
7, 8	5d	6d	6d	5d	6d	5d	5d	4d	5d	5d
9, 10, 11	5d	8d	8d‡	5d	8d	5d	7d	6d	5d	7d
14 and 18	—	10d†	—	—	—	—	9d§	8d	—	—

Standard hooks (and bends) ACI 315-74 (ACI 318-71, Section 7.1)

Bar size	Bend diameter†† Standard bend	Bend diameter†† Ties and stirrups
3, 4, 5	6d	4d
6, 7, 8	6d	**
9, 10, 11	8d	—
14 and 18	10d	—

d = nominal diameter of specimen

*Reference to ACI 318-71 includes 1976 Supplement (Section 3.5.1)
†90 deg bend (supplemental requirement only when specified in purchase order of contract)
‡#11—90 deg bend only
§90 deg when application requires bars to be bent. If bends of greater than 90 deg bend test to be 180 deg
**AASHTO Standard Specification for Highway Bridges (1977 Interim Specification) defines ties and stirrups bend diameter for #6, #7, and #8 bars as 5d.
††Dimensions shown are the finished bend diameters. Pin sizes used by fabricators to obtain these finished bar diameters are generally somewhat smaller than the bend diameter to allow for "springback."

utilized on quality related portions of nuclear projects. The Nuclear Regulatory Commission issued guidelines to be followed in the design, construction, and operation of nuclear power plants. One of these, Regulatory Guide 1.15, deals with the testing of reinforcing bars to be used in nuclear safety related portions of power plants. The Regulatory Guide concern with respect to testing is that ASTM A 615 permits the tensile test specimens to be either of full bar diameter or a reduced diameter. Comparison of test data obtained with both full bar diameter and reduced diameter specimens indicates that the tensile and yield strengths of the full diameter bars may be lower than the values that are obtained using reduced diameter specimens. The variations are generally greater in bar sizes #14 and #18; thus the evaluation of the design margin of safety in the structure may not be conservative if it is based upon test results obtained from reduced diameter specimens, hence the Regulatory Guide requirement of full section testing only.

Regulatory Guide 1.15 also states that the test frequency of ASTM, essentially one test per heat per size, is not adequate considering that heats of steel from the various manufacturers may range from less than 50 tons (45.4 t) to 250 tons (226.8 t) or more. Consequently, Regulatory Guide 1.15 requires tension tests at a maximum frequency of 50 tons (45.4 t) per heat per size.

Regulatory Guide 1.15 does not deal with requirements for additional bend tests nor does it outline any failure/retest criteria. ANSI N45.2.5 includes the 50 ton (45.4 t) test requirement but is equally silent on the bend and retest question. "Code for Concrete Reactor Vessels and Containments (ACI 359-74)" also includes the 50 ton (45.4 t) test requirement, and to some degree attempts to define the failure/retest situation. Unfortunately the criteria that has been developed is not compatible with the current ASTM specification in that a retest and subsequent acceptance is possible under ACI 359-74 but will be rejectable under ASTM A 615-76a.

As an example of this conflict ACI 359-74 does not define a lower limit for test result failures as does ASTM A 615-76a. For example, the yield test result for a particular Grade 60 material is 58,500 psi (404 MPa). The two retests required under ACI 359-74 are 61,500 psi (454 MPa) and 61,000 (421 MPa) which qualifies the material for use on a nuclear power plant project. Under the provisions of ASTM A 615-76a the material would not even be subject to retest since the initial value was more than 1000 psi (6.9 MPa) below the minimum yield, and therefore rejected without any retest.

The 50 ton (45.4 t) test requirement had been interpreted by the owner/engineer/builder as a test to be done by the user either during fabrication or after delivery of the material to the site, hence the term "user tests." This practice has created problems in terms of test result variances between manufacturer and user. The lack of standard acceptance/rejection criteria did not help the resolution of the areas of concern. To help with the resolution of this problem, arrangements may be made with producers of reinforcing bars to test the material to meet the requirements of Regulatory Guide 1.15 and ACI 359-74. This prequalifies the material, thereby eliminating some of the questions.

ACI 318-71 has also added further restrictions to the ASTM reinforcing bar specifications. The 1975 Supplement to ACI 318-71 (Section 3.5.1) includes a requirement relating to the bend testing of reinforcing bars. In essence this section requires the bend test, in some instances, to be made around a smaller pin than is required by ASTM A 615. Table 4 defines ASTM, AASHTO, and ACI 318-71 (1976 Supplement) test bend diameter requirements. The U. S. Army Corps of Engineers, the American Association of State Highway and Transportation Officials, "Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-76)," and a proposed revision to ACI 359-74 require the tighter bend tests.

Bar bends used in practice generally are those established by "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-74)" as shown in Table 4. These dimensions are the finished bend diameter. Pin sizes used by fabricators to obtain these finished bar dimensions are generally somewhat smaller than the bend diameter to allow for "springback."

Most major building projects are designed using ACI 318-71. Many producers can provide the more severe bending requirements of that specification.

Special uses

Spirals—Spirals for column reinforcement may be manufactured from material produced to ASTM A 615-76a specification for plain round and deformed reinforcing bars. Spirals may also be made from cold drawn wire, ASTM A 82-76. The fabrication of spirals and the accompanying spacing devices is generally done by reinforcing bar fabricators. However, all fabricators do not produce spirals. Consequently lead times may run as long as 8 to 10 weeks during times of peak demand.

Bar mats—Bar mats utilize deformed and plain reinforcing bars covered by ASTM A 615, A 616, or A 617 specifications. Fabrication is defined as follows:

"Fabricated Deformed Steel Bar Mats for Concrete Reinforcement" (ASTM A 184-74)—Deformed bar mats are two layers of bars which are assembled at right angles to each other by clipping or welding. Material: ASTM A 615, A 616, A 617 reinforcing bars. The specification outlines the requirements of welding and clipping, the testing of the welds or clips, tolerances, marking, and inspection.

Section 3.5.4 of ACI 318-71 restricts the use of deformed bar mats to the clipped type. However, the 1975 Supplement to this section deletes this exception.

"Welded Steel Plain Bar or Rod Mats for Concrete Reinforcement" (ASTM A 704-74)—Material to be used is defined as ASTM A 615. The mats consist of two layers of bars which are assembled by welding at right angles to each other. The size of the plain rod or bar is ⅝ in. (15.9 mm) maximum and 7/32 in. (5.6 mm) minimum nominal diameter. In contrast to ASTM A 184, assembly is limited to welding only. As in ASTM A 184 the specification defines weld testing, tolerances, marking, and inspection.

It should be pointed out that ASTM A 184-74 (deformed bars) and ASTM A 704-74 (plain bars) both replace ASTM A 184-65, which covered both plain and deformed bars. ACI 318-71 lists ASTM A 184-65 but has not yet included ASTM A 704-74. It should also be noted that there is a difference in the weld shear strength requirements between ASTM A 185 and ASTM A 704. ASTM A 185 requires minimum average weld shear value in pounds of 35,000 times nominal area of larger wire. ASTM A 704 requires only 25,000 times the nominal area of the larger value.

Bar mats are most advantageously used when there is significant repetition; consequently bar mats may not be readily available throughout the country. As with spirals, many of the reinforcing bar fabricating shops do not normally produce bar mats.

NONPRESTRESSED WIRE

Material properties

Wire for concrete reinforcement is available as smooth wire (ASTM A 82) or deformed wire (ASTM A 496) with the properties given in Table 5.

Availability

The smooth wire (ASTM A 82), often used for spirals or welded wire fabric, is generally available throughout the United States and Canada in wire

TABLE 5—MATERIAL PROPERTIES OF SMOOTH AND DEFORMED WIRE

Property	Smooth (ASTM A 82-76)	Deformed (ASTM A 496-72)
Minimum yield strength, psi (MPa)	70,000 (455)	75,000 (517)
Minimum tensile strength, psi (MPa)	80,000 (550)	85,000 (586)
Bend tests:		
W7 and smaller	1*d*	
Larger than W7	2*d*	
D6 and smaller		2*d*
Larger than D6		4*d*

TABLE 6—MINIMUM PROPERTIES OF STEEL WIRES IN WELDED WIRE FABRIC

Wire size	Yield strength,* psi (MPa)	Tensile strength, psi (MPa)	Weld shear strength,† lb (kgf)
Welded smooth wire fabric			
W1.2 and over	65,000 (448)	75,000 (517)	34,000 A_w (24.61 A_w)
Welded deformed wire fabric			
D31 through D4	70,000 (483)	80,000 (552)	20,000 A_w (14.06 A_w)

*The yield strength is determined at a strain of 0.005.
†A_w = nominal area of larger wire in sq in. (sq mm). This requirement applies if, as is generally recommended, the ratio of the area of the smaller wire to the area of the larger wire is at least 0.40 for smooth wire or 0.35 for deformed wire.

sizes from WO.5 [0.080 in. (2.03 mm) diameter] through W31 [0.628 in. (15.95 mm) diameter]. Wire sizes smaller than W1.2 for welded wire fabric are always galvanized. Smooth wire used in the manufacture of welded wire fabric for concrete reinforcement is generally limited to a minimum size of W1.2 [0.124 in. (3.51 mm) diameter].

Spirals fabricated from cold drawn wire are subject to the same availability and lead time requirements as spirals using ASTM A 615 plain material.

Deformed wire (ASTM A 496) most commonly used in the manufacture of deformed welded wire fabric is normally available in wire sizes D4 through D31. Although some deformed welded wire fabric uses D3 wire, ACI 318-71 limits the minimum deformed wire size to D4.

WELDED WIRE FABRIC

Material properties

Welded wire fabric, the material and the manufacture of, is covered by "Welded Steel Wire Fabric for Concrete Reinforcement" (ASTM A 185) for fabric made from smooth wire and "Welded Deformed Steel Wire Fabric for Concrete Reinforcement" (ASTM A 497) for fabric made from deformed wire. These two specifications define the minimum weld shear strength; tension, bend, and weld shear test methods and apparatus; widths, permissible variations, packaging, and identification requirements. Table 6 summarizes the minimum properties of steel wires in welded wire fabric.

Since the yield strength is measured at 0.005 strain, ACI 318-71 allows the use of the material only as though it had a specified yield of 60,000 psi (414 MPa), unless welded wire fabric is specified and furnished with yield strength measured at 0.0035 strain.

The nomenclature used to describe the type and size of welded wire fabric, sometimes mistakenly referred to as mesh, is presently in a period of transition from the old designation to the new. Previously fabric was referred to by using the longitudinal and transverse wire spacing and the respective wire gage as: 6 x 6-4 x 4 or 6-6 4/4 which denoted wire spacings at 6 in. (152 mm) each way using 4 gage wire each way.

The new designations show spacing of longitudinal and transverse wires in the same manner. The gage has been replaced by an identification W or D for smooth or deformed wire and the area in hundredths of a square inch as 4 x 4-W5 x W5 indicates longitudinal and transverse wire spacing at 4 in. (102 mm) with smooth wire having a cross-sectional area of 0.050 in.^{2} (32.3 mm^2) for both the longitudinal and transverse wire.

Currently both ways of identifying fabric styles are in use, and will continue to be until the transition to the new method has been completed. Specifiers of fabric for specific design requirements generally use the new designation while some specifiers and users of the common lighter building fabric styles still use the old designation.

Availability

As noted in the previous sections on nonprestressed wire, welded wire fabric for concrete reinforcement is generally available with minimum wire sizes of W1.2 for smooth fabric and D4 for deformed fabric.

Minimum quantity requirements

Welded wire fabric becomes more efficient and economical as the amount of repetition in reinforcement increases. Economy is governed by the manufacturing process and by the industry practice of carrying certain common welded wire fabric item in stock or inventory.

Stock items

Certain items of welded smooth wire fabric are carried in stock either at the producing mills or

TABLE 7—COMMON STOCK STYLES OF WELDED WIRE FABRIC

Steel designation		Steel area, sq in. per ft‡		Weight (approximate) lb per 100 sq ft§
New designation (by W-number)	Old designation (by steel wire gage)	Longitudinal	Transverse	
Rolls				
6 x 6-W1.4 x W1.4	6 x 6-10 x 10	0.029	0.029	21
6 x 6-W2.0 x W2.0	6 x 6-8 x 8*	0.041	0.041	30
6 x 6-W2.9 x W2.9	6 x 6-6 x 6	0.058	0.058	42
6 x 6-W4.0 x W4.0	6 x 6-4 x 4	0.080	0.080	58
4 x 4-W1.4 x W1.4	4 x 4-10 x 10	0.043	0.043	31
4 x 4-W2.0 x W2.0	4 x 4-8 x 8*	0.062	0.062	44
4 x 4-W2.9 x W2.9	4 x 4-6 x 6	0.087	0.087	62
4 x 4-W4.0 x W4.0	4 x 4-4 x 4	0.120	0.120	85
Sheets				
6 x 6-W2.9 x W2.9	6 x 6-6 x 6	0.058	0.058	42
6 x 6-W4.0 x W4.0	6 x 6-4 x 4	0.080	0.080	58
6 x 6-W5.5 x W5.5	6 x 6-2 x 2†	0.110	0.110	80
4 x 4-W4.0 x W4.0	4 x 4-4 x 4	0.120	0.120	85

*Exact W-number size for 8 gage is W2.1.
†Exact W-number size for 2 gage is W5.4.
‡0.01 sq in./ft = 21.17 sq mm/m
§1 lb/100 sq ft = 4.88 kgf/100 sq m

warehousing points. While practice varies somewhat with different manufacturers and localities, the items listed in Table 7 are usually available. Typical roll widths and lengths are given in Table 8.

Nonstock items

It is often desirable to order welded wire fabric sheets or rolls specifically produced to meet the reinforcing requirements and dimensions for individual projects.

The minimum quantity requirements for nonstock items are governed by the manufacturing process requirements.

Quantity requirements vary with different producers but the following examples illustrate the general requirements:

1. Longitudinal spacing, wire size, and fabric width changes require 10 ton (9.07 t) to 20 ton (18.14 t) quantities per item.

2. Transverse wire spacing and size, side and end overhangs, and length changes require 2 ton (1.81 t) to 5 ton (4.54 t) quantities per item.

3. The average item weight for the total quantity ordered for each nonstock item should be approximately 15 tons (13.61 t).

For nonstock welded wire fabric items the following guidelines will lead to the greatest production economies:

1. The most important factor affecting economy is to minimize the number of different longitudinal wire spacings in any group of items. The number of spacings required can be reduced by varying longitudinal wire sizes to obtain the required steel areas per foot of width.

2. The second most important factor is controlling the number of different wire sizes required. A change in transverse wire spacings is relatively easy. Vary the transverse wire spacings and use a minimum number of transverse wire sizes to obtain the required transverse steel areas.

Welded wire fabric may also be used as stirrup and tie reinforcement in beams and columns. This practice is more prevalent in European countries than in the United States. There is special equipment available to fabricate fabric into many stirrup and tie configurations. Obviously, significant duplication is necessary to effectively utilize fabric in this application.

PRESTRESSED REINFORCEMENT

Introduction

Prestressing steel is an active reinforcement. Up to now this report has dealt with passive reinforc-

TABLE 8—TYPICAL ROLL WIDTHS AND LENGTHS OF WELDED SMOOTH WIRE FABRIC

United States except West Coast	60 in. x 150 ft-0 in. (1524 mm x 45.7 m)
United States—West Coast	84 in. x 150 ft-0 in. (2134 mm x 45.7 m) 84 x 200 ft-0 in. (2134 mm x 61.0 m)
Canada	60 in. x 200 ft-0 in. (1524 mm x 61.0 m) 72 in. x 200 ft-0 in. (1829 mm x 61.0 m)

TABLE 9—MINIMUM PRESTRESSING REINFORCEMENT PROPERTIES

ASTM A 421 wire

Size, in.	Yield strength, psi†‡		Tensile strength, psi		Elongation, percent
	Type BA	Type WA	Type BA	Type WA	10 in. (254 mm) gage length
0.192	*	200,000	*	250,000	4.0
0.196	192,000	200,000	240,000	250,000	4.0
0.250	192,000	192,000	240,000	240,000	4.0
0.276	*	188,000	*	235,000	4.0

ASTM A 416 strand

Size	Yield strength, psi†	Tensile strength, psi	Elongation, percent 24 in. (610 mm) gage length
All sizes:			
Grade 270	229,500	270,000	3.5
Grade 250	212,500	250,000	3.5

*Sizes not commonly furnished in Type BA wire.

†Minimum yield measured at 1 percent extension under load except as noted below under low relaxation properties. (Minimum yield/tensile is 80 percent for ASTM A 421 and 85 percent for ASTM A 416.)

‡100,000 psi = 689 MPa.

ing system. The term "active" describes the prestressing system constantly applying a force to the structural element regardless of the external loads on that element. Because the magnitude of force which the prestressing system delivers to the structural element is of a prime importance, the mechanism of transferring that force from the prestressing steel to the structural element is a major consideration. The transfer mechanism selected for any prestressing technique will have a direct bearing on material property requirements. Pretensioned prestress reinforcement requires that the force first be introduced in the prestressing steel and maintained while concrete is placed around the prestressing steel and cured. At this time the force is then transferred to the surrounding concrete which is bonded to the prestressing steel. Special material properties may be required for the bond transfer mechanism, usually this is accomplished by using seven-wire strand.

Prestressing steel for post-tensioned members includes a wider spectrum of materials, i.e., wire, bar, or strand are used interchangeably in post-tensioning members, depending upon the economics of the prestressing steel, the anchor system, and the considerations for the member being prestressed. In post-tensioning systems, the force is transferred from the prestressing steel through some mechanical device to an anchor, and then usually through a bearing plate to the member to be prestressed. There is a wide variety of mechanisms used to transfer the force from the prestressing steel. The most commonly used ones in this country are wedge grips, threaded connectors, or upset ends (button heads) on the prestressing steel. The performance of the anchor system and its effect on the prestressing steel may affect the performance of the post-tensioning system to a greater degree than variables in the prestressing steel. The prestressing steel may be bonded to the surrounding concrete after the prestressing force is applied by injecting grout into the void, or it may be left unbonded and coated with a corrosion inhibitor.

Material properties

Reinforcement for prestressed concrete may consist of high strength wire as defined by "Uncoated Stress-Relieved Wire for Prestressed Concrete" (ASTM A 421-76), strand as defined by "Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete" (ASTM A 416-74), or bar as defined by "Uncoated High-Strength Steel Bar for Prestressing Concrete" (ASTM A 722-75).

Wire produced under ASTM A 421 varies in minimum ultimate tensile strength depending on wire size. It may be used in applications that require cold-end deformations for anchorage, Type BA (Button Anchorage) and for application that require wedge anchorage of the ends, Type WA (Wedge Anchorage) with no cold-end deformation. The specification also provides under Supplement I for low relaxation wire and relaxation testing for the product.

Strand for prestressed application is seven-wire type consisting of one center wire and six helically placed outer wires with a uniform pitch of not less than 12 nor more than 16 times the nominal strand diameter. Material furnished under ASTM A 416

TABLE 10—SUMMARY OF ASTM A 722-75 REQUIREMENTS FOR UNCOATED HIGH-STRENGTH STEEL BAR FOR PRESTRESSING CONCRETE

Mechanical properties*	Type I (plain bars)	Type II (deformed bars)
Minimum ultimate tensile strength	150,000 psi (1034 MPa)	150,000 psi (1034 MPa)
Minimum yield	85 percent minimum ultimate tensile	80 percent minimum ultimate tensile
Strain at yield (extension under load method)	0.7 percent	0.7 percent
Elongation (gage length 20 bar diameters)	4 percent	4 percent
Size ranges [increments of ⅛ in. (3.2 mm)]	¾ in. (19.0 mm) to 1⅜ in. (34.9 mm)	⅝ in. (15.9 mm) to 1⅜ in. (34.9 mm)
Supplementary requirements (Apply only when specified by purchaser)		
Bending (full size), in. (mm)	$6d$: ⅝, ¾, and 1 (15.9, 19.0, 25.4) $8d$: 1¼ and 1⅜ (31.75, 34.9)	$6d$: ⅝, ¾, and 1 (15.9, 19.0, 25.4) $8d$: 1¼ and 1⅜ (31.75, 34.9)
Mechanical coupling		Coupling requirements when deformation acts as threads
Reduction of area	20 percent minimum	20 percent minimum

*Bars to be cold-stressed to not less than 80 percent minimum ultimate strength and then stress-relieved to produce prescribed mechanical properties.

is available in two grades, 250 and 270, with minimum ultimate strengths of 250,000 psi (1724 MPa) and 270,000 psi (1861 MPa), respectively, based on nominal area of the strand. Low relaxation strand is also available under Supplement I.

Table 9 defines the properties of prestressing wire and strand (ASTM A 421 and A 416).

Material such as high strength alloy steel bars may also be used as a prestressing reinforcement. ACI 318-71 requires in Section 3.5.10 that such bars be proof-stressed to 85 percent of minimum guaranteed tensile strength. Following proof-stressing the bars should be subject to stress-relieving heat treatment to produce the following properties determined by full section tests.

Yield strength (0.2 percent offset)	$0.85\ f_{pu}$
Elongation at rupture in 20 diameters	4 percent
Reduction in area at rupture	20 percent

where

f_{pu} = ultimate strength of prestressing steel.

In late 1975, ASTM approved ASTM A 722-75. This specification is briefly summarized in Table 10. It is anticipated that the current ACI 318-71 requirements on bars used in prestressing applications will be deleted and the provisions of ASTM A 722-75 will be included in the ACI Building Code.

Note for low relaxation wire and strand

1. Relaxation loss after 1000 hr not more than 2.5 percent when initially loaded to 70 percent of specified minimum tensile strength or not more than 3.5 percent when loaded to 80 percent of specified minimum tensile strength at 68 F (20 C).

2. For low relaxation wire or strand, minimum yield measured at 1 percent extension under load

TABLE 11—SIZES OF PRESTRESSING STRAND AVAILABLE (ASTM A 416)

Nominal diameter	Grade 250	Grade 270
¼ in. (0.250 in., 6.35 mm)	A	NA
5/16 in. (0.313 in., 7.9 mm)	A	NA
⅜ in. (0.375 in., 9.5 mm)	A	A
7/16 in. (0.438 in., 11.1 mm)	A	A
½ in. (0.500 in., 12.7 mm)	A	A
(0.600 in. 15.2 mm)	A	A

A—Available; NA—Not available

shall not be less than 90 percent of specified minimum tensile strength.

Availability

Prestressing strand (ASTM A 416) is generally available throughout the country in the sizes noted in Table 11. Prestressing wire or bar are generally available as part of prestressing systems which include complete tendons and anchorage devices.

CONCLUSION

This report has identified the properties and discussed the availability of the different types and sizes of steel reinforcement for concrete structures. The properties described are those specified for the as-delivered products, and the designer should note and make appropriate arrangements where particular design specifications are more restrictive than the ASTM specifications.

SOURCES

Standards and specifications

1. American Association of State Highway and Transportation Officials, 444 N. Capitol Street N.W., Suite 225, Washington, D. C. 20045

Standard Specifications for Highway Bridges, Eleventh Edition, 1973 Interim Specification Bridges, 1976

Designation M-31-77I, Deformed and Plain Billet Steel Bars for Concrete Reinforcement

2. American Concrete Institute, P. O. Box 19150, Redford Station, Detroit, Mich. 48219

ACI 315-74	Manual of Standard Practice for Detailing Reinforced Concrete Structures
ACI 318-74	Building Code Requirements for Reinforced Concrete (including 1976 Supplement)
ACI 349-76	Code Requirements for Nuclear Safety Related Concrete Structures
ACI 359-74	Code for Concrete Reactor Vessels and Containments Section III, Division 2 of the ASME Boiler and Pressure Vessel Code

3. American National Standards Institute, Inc., 1430 Broadway, New York, New York 10018: Supplementary Quality Assurance Requirements for Installation, Inspection and Testing of Structural Concrete and Structural Steel During the Construction Phase of Nuclear Power Plants (ANSI N45.2.5—1974)

4. American Society for Testing and Materials, 1916 Race St., Philadelphia, Pennsylvania 19103

A 82-76	Cold-Drawn Steel Wire for Concrete Reinforcement
A 184-74	Fabricated Deformed Steel Bar Mats for Concrete Reinforcement
A 185-73	Welded Steel Wire Fabric for Concrete Reinforcement
A 370-76	Standard Methods and Definitions for Mechanical Testing of Steel Products
A 416-74	Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete
A 421-76	Uncoated Stress-Relieved Wire for Prestressed Concrete
A 496-72	Deformed Steel Wire for Concrete Reinforcement
A 497-72	Welded Deformed Steel Wire Fabric for Concrete Reinforcement
A 615-76a	Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
A 616-76	Rail-Steel Deformed and Plain Bars for Concrete Reinforcement
A 617-76	Axle-Steel Deformed and Plain Bars for Concrete Reinforcement
A 704-74	Welded Steel Plain Bar or Rod Mats for Concrete Reinforcement
A 706-76	Low-Alloy Steel Deformed Bars for Concrete Reinforcement
A 722-75	Uncoated High-Strength Steel Bar for Prestressing Concrete

5. Associated Reinforcing Bar Producers—Concrete Reinforcing Steel Institute, 180 North LaSalle St., Room 2110, Chicago, Ill. 60601: Reinforcing Bar Testing, 1974

6. Concrete Reinforcing Steel Institute, 180 North LaSalle St., Room 2110, Chicago, Ill. 60601: Manual of Standard Practice, 22nd Edition, 1976 First Printing

7. Nuclear Regulatory Commission (U. S. Atomic Energy Commission), Washington, D. C. 20555: Regulatory Guide 1.15, Testing of Reinforcing Bars for Category I Concrete Structures

8. Wire Reinforcement Institute, Inc., 7900 Westpark Dr., McLean, Virginia 22101: Welded Wire Fabric—Manual of Standard Practice, 1972

9. American Welding Society, Inc., 2501 N.W. 7th St., Miami, Florida 33125: Reinforcing Steels Welding Code (AWS D12.1-75)

References

1. Cowell, W. L., "Dynamic Tests of Concrete Reinforcing Steels, *Technical Report* No. R394, U. S. Naval Civil Engineering Laboratory, Port Hueneme, 1965, 34 pp. (Available as AD 622 544, National Technical Information Services).

2. "Design of Structures to Resist the Effects of Atomic Weapons—Strength of Materials and Structural Elements," *Manual* No. EM 1110-345-414, U. S. Army Corps of Engineers, 1957, 64 pp.

3. Seiss, C. P., "Behavior of High Strength Deformed Reinforcing Bars Under Rapid Loading," Department of Civil Engineering, University of Illinois, Urbana, 1962.

4. Crum, Ralph G., "Tensile Impact Tests for Concrete Reinforcing Steels," ACI JOURNAL, *Proceedings* V. 56, No. 1, July 1959, pp. 59-61.

5. Rice, Paul F., and Gustafson, David P., "Grade 80 Reinforcing Bars and ACI 318-71," ACI JOURNAL, *Proceedings* V. 73, No. 4, April 1976, pp. 199-206.

6. McDermott, John F., "Reinforcing Steel," *Reinforced Concrete Engineering,* Boris Bresler, ed., John Wiley and Sons, New York, 1974, pp. 42-64.

This report was submitted to letter ballot of the committee, which consists of 16 members; 15 members returned their ballots all of whom voted affirmatively.

INDEX

CONVERSION FACTORS—U.S. CUSTOMARY TO SI (METRIC)*

To convert from	to	multiply by
Length		
inch	millimeter (mm)	25.4E†
foot	meter (m)	0.3048E
yard	meter (m)	0.9144E
mile (statute)	kilometer (km)	1.609
Area		
square inch	square centimeter (cm^2)	6.451
square foot	square meter (m^2)	0.0929
square yard	square meter (m^2)	0.8361
Volume (capacity)		
ounce	cubic centimeter (cm^3)	29.57
gallon	cubic meter (m^3)‡	0.003785
cubic inch	cubic centimeter (cm^3)	16.4
cubic foot	cubic meter (m^3)	0.02832
cubic yard	cubic meter (m^3)‡	0.7646
Force		
kilogram-force	newton (N)	9.807
kip-force	newton (N)	4448
pound-force	newton (N)	4.448
Pressure or stress (force per area)		
kilogram-force/square meter	pascal (Pa)	9.807
kip-force/square inch (ksi)	megapascal (MPa)	6.895
newton/square meter (N/m^2)	pascal (Pa)	1.000E
pound-force/square foot	pascal (Pa)	47.88
pound-force/square inch (psi)	kilopascal (kPa)	6.895

To convert from	to	multiply by
Bending moment or torque		
inch-pound-force	newton-meter (Nm)	0.1130
foot-pound-force	newton-meter (Nm)	1.356
meter-kilogram-force	newton-meter (Nm)	9.807
Mass		
ounce-mass (avoirdupois)	gram (g)	28.34
pound-mass (avoirdupois)	kilogram (kg)	0.4536
ton (metric)	megagram (Mg)	1.000E
ton (short, 2000 lbm)	megagram (Mg)	0.9072
Mass per volume		
pound-mass/cubic foot	kilogram/cubic meter (kg/m^3)	16.02
pound-mass/cubic yard	kilogram/cubic meter (kg/m^3)	0.5933
pound-mass/gallon	kilogram/cubic meter (kg/m^3)	119.8
Temperature§		
deg Fahrenheit (F)	deg Celsius (C)	$t_C = (t_F - 32)/1.8$
deg Celsius (C)	deg Fahrenheit (F)	$t_F = 1.8t_C + 32$

*This selected list gives practical conversion factors of units found in concrete technology. The reference source for information on SI units and more exact conversion factors is "Standard for Metric Practice" ASTM E 380. Symbols of metric units are given in parentheses.

†E Indicates that the factor given is exact.

‡ One liter (cubic decimeter) equals 0.001 m^3 or 1000 cm^3.

§ These equations convert one temperature reading to another and include the necessary scale corrections. To convert a difference in temperature from Fahrenheit degrees to Celsius degrees, divide by 1.8 only, i.e., a change from 70 to 88 F represents a change of 18 F or 18/1.8 = 10 C deg.