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Guide to Materials Selection for Concrete Repair

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Guide to Materials Selection for Concrete Repair

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Guide to Materials Selection for Concrete Repair

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This document provides guidance on the selection of materials for concrete repair. An overview of the important properties of repair materials is presented as a guide for making an informed selection of the appropriate repair materials for specific applications and service conditions.

Keywords: cementitious; cracks; epoxy; materials; methacrylate; polymer; polyurethane; repair; surface sealer; silica fume; test methods; waterproofing.

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

Concrete is inherently a durable material, but its durability under any given set of exposure conditions varies with concrete mixture proportions; the presence and positioning of reinforcement; and the detailing, placing, finishing, curing, and protection it receives. In service, it may be exposed to conditions of abrasion, moisture cycles, cycles of freezing and thawing, temperature fluctuations, reinforcement corrosion, and chemical attack, resulting in deterioration and potential reduction of its service life.

As the concrete industry develops and grows, concrete repair is frequently required; however, with the increasing number and age of concrete structures, frequent deferral of maintenance, and increased public awareness of deterioration and maintenance needs, repair is becoming a major focus of design and construction activities. Although concrete repair is traditionally as much an art as a science, engineers and contractors typically do not receive much formal training in techniques for repair and the performance of repair materials applied to concrete. Personal experience is beneficial, but takes time to accumulate and can be costly in terms of failed repairs. Although this is changing, there is still too little information available to reliably predict the serviceability and durability of repairs. Concrete repairs that fail prematurely result in economic loss and usually require additional repairs.

Due to a greatly expanded repair market, new materials and repair methods are being introduced at an increasing rate to the construction market. At the same time, the changing environmental and building codes and other regu-

lations, many existing, well-proven products are being reformulated into essentially new products that have limited track records. The user might not be informed of these changes.

It is often difficult for a specifier to find the appropriate data to systematically evaluate a product for a given repair situation. Often, test data are unavailable or, if available, are either not presented in useful or appropriate terms or presented in a manner that makes comparison with other competing materials difficult. One example is the use of nonstandard or modified test methods.

Although there are many competent repair materials available commercially, there are also unsubstantiated claims of suitability and success. Even the highest-quality materials do not perform as expected if they are used inappropriately.

ACI 546R is the first ACI publication devoted entirely to the subject of concrete repair. Its principle emphasis is on techniques for concrete repair with limited information on selecting repair materials. The physical properties of repair materials govern their performance in service and, as a result, the appropriate selection of these materials for a given repair is at least as important to a successful, long-lasting repair as is using the proper procedures and workmanship. This guide is the second in a series of documents prepared by Committee 546 to aid the user in specifying and executing typical concrete repairs.

1.2—Essential steps of concrete repair

The success of concrete repairs depends on determining the cause and extent of concrete distress or deterioration and developing a repair strategy to address the problem. Typical steps in a systematic repair are to:

- a) Conduct a condition survey with a scope consistent with the perceived condition of the structure and the owner's repair objectives, performed by qualified individuals, to document and evaluate visible and concealed deterioration, distress, defects, and damage, as well as potential future deterioration and distress;
- b) Determine the cause of the damage or deterioration necessitating the repair—for example, mechanical damage such as impact or abrasion; design, detailing, or construction deficiencies; chemical damage such as alkali-aggregate reaction; physical damage related to cycles of freezing and thawing or thermal movements; corrosion of the steel reinforcement caused by improper placement; carbonation of the concrete; or chloride ingress into the concrete to the reinforcing steel;
- c) Assess the application and service conditions to which the concrete repair material is, or will be, exposed;
- d) Determine the repair objectives, including desired service life;
- e) Select a repair strategy, including consideration of an appropriate protection system in conjunction with future maintenance, in terms of what is required to preserve or protect the structure and repairs, and what actual maintenance is likely to be available.

Once the concrete to be repaired is evaluated and the cause of distress established, details of the proposed repair are developed. This includes evaluating and determining the

required physical properties of repair materials, followed by the appropriate selection of available repair materials. Selection is usually based on the ability of the material to conform to repair constraints and objectives as defined in this guide, including consideration of cost and availability.

The repair is then implemented, including protective systems if designed as part of the repair. Refer to ACI 546R, where these steps are discussed in further detail.

1.3—Objective

The objective of this guide is to provide guidance for the materials selection for concrete repair, including:

- a) Identification of common repair materials;
- b) Discussion of relevant material properties;
- c) Lists and discussion of test procedures for measuring these properties;
- d) Recommendations of minimum test values or performance levels;
- e) Discussion of the importance of specific material properties for various repair applications and service environments.

1.4—Scope

This guide discusses material selection for several types of repairs and materials used in their repair:

- a) Concrete replacements, categorized on the basis of the depth and orientation of repair;
- b) Overlays, categorized on the basis of their thickness;
- c) Crack repairs, categorized on the basis of crack stability, crack width, and other service conditions;
- d) Surface sealers and traffic-bearing elastomeric coatings, categorized on the basis of their water and chloride ion permeability;
- e) Anti-carbonation coatings, categorized on the basis of their carbon dioxide diffusion;
- f) Reinforcing steel coatings, embedded galvanic anodes, concrete bonding agents and procedures, crystalline pore blockers, and surface-applied, penetrating corrosion inhibitors, categorized on their ability to alter and improve various concrete properties.

1.4.1 Concrete replacement and overlay materials discussed in this guide include:

- a) Portland or blended cement-based mortar and concrete;
- b) Portland or blended cement-based silica-fume mortar and concrete;
- c) Portland or blended cement-based polymer-cement mortar and concrete;
- d) Magnesium-ammonium-phosphate-cement mortar and concrete;
- e) Polymer-based mortar and concrete.

1.4.2 Crack repair materials discussed in this guide include:

- a) Epoxy resin;
- b) Methacrylate resin;
- c) Polyurethane chemical grout;
- d) Polyurethane sealant;
- e) Silicone sealant;
- f) Silyl-terminated polyether sealant;
- g) Polysulfide sealant;

- h) Flexible epoxy resin;
- i) Polyurea;
- j) Strip and seal systems, including preformed flexible sheets;
- k) Polymer grout;
- l) Polymer-cement grout;
- m) Cementitious grout.

1.4.3 Surface sealer materials discussed in this guide include:

- a) Silanes;
- b) Siloxanes;
- c) Acrylics;
- d) Epoxies;
- e) Linseed oil.

1.4.4 Anti-carbonation coating materials discussed in this guide include:

- a) Acrylics;
- b) Methacrylate coating;
- c) Polymer-modified cementitious materials.

1.4.5 Traffic-bearing elastomeric coating materials discussed in this guide include:

- a) Polyurethane systems;
- b) Polyurethane-epoxy composite systems;
- c) Polyurea systems.

1.4.6 Reinforcing steel coating materials discussed in this guide include:

- a) Modified cementitious materials;
- b) Epoxies;
- c) Zinc-rich products.

1.4.7 Concrete bonding materials and procedures discussed in this guide include:

- a) Preparing a clean, dry substrate;
- b) Preparing a saturated surface-dry substrate;
- c) Prepare a saturated surface-dry substrate with scrub coat of paste from replacement material;
- d) Acrylic bonding agents;
- e) Epoxy bonding agents.

1.4.8 Crystalline pore blockers and surface-applied, penetrating corrosion inhibitors—These are proprietary products with undisclosed ingredients. This guide discusses these materials on a generic performance basis.

1.4.9 Summary tables of test methods and test values—**Tables 3.8a** and **3.8b** present summaries of available test methods and typical test values for concrete replacement and overlay materials; **Table 4.7** presents a selection guide for these materials. **Tables 5.7a** and **5.7b** present summaries of available test methods and typical test values for crack repair materials; **Table 6.14** presents a selection guide for these materials. **Tables 7.5a**, **7.5b**, and **7.5c** present summaries of available test methods and typical test values for surface sealers, anti-carbonation coatings, and traffic-bearing elastomeric coatings; and **Tables 8.5a** and **8.5b** present selection guides for surface sealers and anti-carbonation coatings.

1.4.10 Safety cautions—Repair material specifiers and users should be aware that many repair materials have to be handled with care to avoid potential harm to workers and the environment. Health and safety practices have to be established appropriate to the specific circumstances involved

with material use. The applicability of all regulatory limitations have to be determined when selecting repair materials, and material selection and use have to comply with all applicable laws and regulations.

1.4.11 Special repair and service environments—This guide covers concrete repair materials for common types of concrete construction. Special repair and service environments may require repair materials with enhancement of particular properties. For the repair of environmental or mass-concrete structures, underwater repairs, and other special repair and service environments, refer to the recommendations of other ACI publications (including those of ACI Committee 350 and **ACI 546.2R**), industry organizations, and material manufacturers for specific guidance in repair material selection.

1.5—Current industry issues and concerns

Appendix A includes a discussion of a number of current industry issues and concerns, including:

- a) Material test methods and reporting of test data;
- b) Curing of repair materials and manufacturers' reported test results;
- c) Product limitations and warnings;
- d) Standardized industry acceptance;
- e) Repair material bond;
- f) Corrosion reduction;
- g) Structural repairs; and
- h) Ongoing developments.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

There are no notations used in this guide.

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, “ACI Concrete Terminology,” <http://www.concrete.org/Tools/ConcreteTerminology.aspx>. Definitions provided herein complement that source.

basic portland-cement concrete—a composition of portland cement, fine and coarse aggregates, and water.

concrete replacement—the removal and replacement of damaged or deteriorated concrete, including partial-depth as well as full-depth repairs that are sometimes informally called patches.

electrical resistivity—the resistance of a material to the flow of electrical charge in the presence of a voltage potential between two points.

permeability—the ability of a material to transmit or resist the penetration of water and water-borne chemicals. This definition differs from that in ACI Concrete Terminology in that the ability to transmit or the resist the penetration of gases is not included.

surface sealer—material applied to the concrete surface to reduce moisture and chemical penetration into the concrete member. Surface sealers may achieve limited penetration into uncracked concrete, but are either too brittle to apply in a coating too thin to bridge moving cracks or cracks that

may form in the concrete after the material application. Surface sealers are differentiated from **sealers**, as defined in ACI Concrete Terminology, in that surface sealers may or may not prevent the penetration of gaseous media, be colorless, or be absorbed by the concrete, and commonly are visible on the concrete surface.

traffic-bearing elastomeric coating—material applied to the concrete surface to greatly reduce moisture and chemical penetration to the concrete member and that is suited for exposure to pedestrian and vehicular traffic. Elastomeric coatings have some flexibility so they are capable of bridging narrow cracks that experience some movement and some cracks that might form in the concrete after the elastomeric coating application.

volume stability—initial and long-term changes in the linear dimensions or volume of a material after placement.

working time—elapsed time from completion of mixing until the material becomes too stiff or otherwise unusable.

CHAPTER 3—PROPERTIES OF CONCRETE REPLACEMENT AND OVERLAY MATERIALS AND THEIR IMPORTANCE

3.1—General

Compatibility between the properties of concrete replacement and overlay materials and the properties of the intended substrate is an important consideration. For example, in many concrete replacement applications, the properties of repair materials, such as the coefficient of thermal expansion and creep, should be similar to those of the substrate. In contrast, the success of many crack repair applications depends on repair materials that have significantly different properties, such as high elasticity and low modulus of elasticity, from that of the substrate, and which will perform better than the base concrete in the service environment. Regardless, it is necessary to identify the repair material properties and the substrate properties before an approach to the repair is determined (McDonald et al. 2002).

Many properties of replacement materials, overlay materials, and the existing concrete are time-dependent. In cases where material properties are specified, the corresponding age of the materials should be noted. A conclusion from Alberta Transportation's 1987 concrete replacement testing program was that the durability of concrete replacement materials correlated better with long-term physical properties—for example, 1 year or more after placement, rather than shorter-term physical properties (24-hour and 28-day properties) (Gurjar 1987).

Most test methods cited in this guide are performed at standard conditions at room temperature in many cases, with standard-sized specimens, so reported properties may not reflect the actual properties of the repair material in various-sized repairs in service conditions.

Chapter 3 discusses properties of concrete replacement and overlay materials, and test methods used to evaluate them. Some test methods are not specifically applicable for certain replacement or overlay materials, or replacement or overlay installations, but are useful for comparison. Because

descriptions of the various test methods are brief, referenced standards should be reviewed for details. Material manufacturers should provide test data based on ASTM standards and other standardized test methods. ACI 364.3R and ICRI 320.3R describe many relevant properties and appropriate modifications to standardized test methods suitable for cementitious replacement materials. Refer to Appendix A for further discussion regarding modifications to standard test methods.

3.2—Volume stability

Volume stability properties affect compatibility of the repair material with the substrate concrete. Substrate concrete is usually relatively stable, with minimal residual creep and shrinkage deformations. The substrate concrete, however, may experience some volume instability for various reasons, including thermal expansion and contraction from seasonal environmental changes. It is desirable for any shrinkage or expansion of the repair material to occur before the repair material has reached its final set, while creep is high. If this is not possible with some repair materials, consideration should be given to accommodating differential movements in some manner in the repair design, such as use of control joints, curing, avoidance of reentrant corners, avoidance of high length-to-width ratio configurations, or treatment of anticipated cracks.

Most cementitious materials undergo early shrinkage within the first few hours to days after application. Non-cementitious materials, such as those with polymeric binders, tend to be more stable with little or no shrinkage after hardening. These materials, however, are subject to greater volume changes due to temperature variations. Significant changes in repair material volume can cause high shear stresses at the interface, debonding from the substrate concrete, and cracking of the repair material. Stresses created in the repair material by restrained contraction and expansion may be reduced by using repair materials with a lower modulus of elasticity or a higher rate of creep. Expansion of the repair material may be resisted by providing restraint or confinement, such as by keying it into the substrate concrete. Cracking of the repair material to relieve restrained shrinkage should be anticipated, and further repairs may be needed.

Six test methods are used to evaluate volume stability of concrete, mortar, and cementitious materials:

- 1) ASTM C157
- 2) ASTM C157, as modified by ICRI 320.3R
- 3) ASTM C596
- 4) ASTM C806
- 5) ASTM C827
- 6) ASTM C1581

ASTM C157, ASTM C596, and ASTM C806 are test methods that involve monitoring the length of test specimens over time under different curing conditions. A restraining cage with an embedded steel rod is used in ASTM C806 to restrain the specimen expansion. ASTM C827 is a test method that involves monitoring the height of cylindrical test specimens until the specimens harden. The procedure that involves measuring the strains and observing

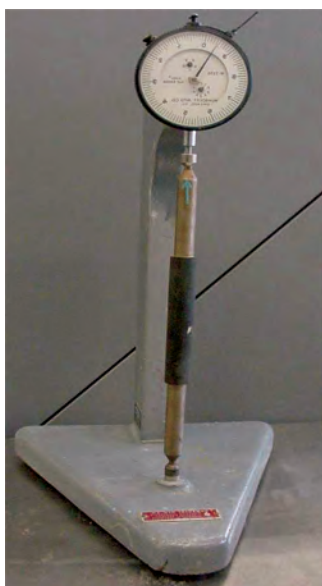


Fig. 3.2.1—Length comparator with standard calibrating rod (ASTM C157). (Photo courtesy of BASF Construction Chemicals, LLC.)

cracking in donut-shaped specimens with inner steel rings. The primary use of these test methods is to provide a relative comparison of various materials when tested in the same fashion. Actual shrinkage experienced in the field will likely differ from shrinkage reported from tests; harsh laboratory test conditions sometimes yield higher shrinkage than that which actually occurs in the field.

3.2.1 ASTM C157—The curing and comparator reading regimen in this test method is not applicable for repair mortars and concretes. The test specimens are approximately 11-1/4 in. (285 mm) long and vary in cross section from 1 in. (25 mm) square for mortar specimens to 3 or 4 in. (75 or 100 mm) square for concrete specimens. Initially, the test specimens are stored in a moist room for 24 hours, demolded, and placed in lime-saturated water for a minimum of 15 minutes for mortar specimens and 30 minutes for concrete specimens. Initial length comparator measurements are taken. The specimens are stored in lime-saturated water until they have reached an age of 28 days, when another set of length comparator measurements are made. After these measurements, the specimens are stored in either lime-saturated water or in a drying room, and length comparator measurements are taken at 4, 7, 14, and 28 days (drying room storage only), and 8, 16, 32, and 64 weeks, including the initial 28-day curing period. The length change, in percent, at any age is calculated. Typical shrinkage strains range from 0.02 percent expansion to -0.12 percent shrinkage. The length comparator setup is shown in Fig. 3.2.1.

Neither curing regimen is representative of field conditions for most repair mortars because the initial length-comparator measurement is made at 24 hours, neglecting volume changes that occur within the first 24 hours; extensive wet curing is used, and the specimens are not restrained. In field conditions, bonding repair materials to the substrate concrete provides restraint to shrinkage and an increased

exposed surface area, such as shallow or long slender repair geometries, can significantly increase shrinkage.

When testing shrinkage-compensating mortars, unrealistic curing conditions and inappropriate comparator reading schedules could lead to misleadingly low values of drying shrinkage. It is critical that the demolding time, curing conditions, and comparator reading schedule are understood when interpreting test results. For example, if the initial measurement is recorded while the material is still expanding, the ultimate drying shrinkage appears less than it actually is; the net length change (expansion less shrinkage) during the test, therefore, should be used as the value for drying shrinkage.

The specimen size has a significant effect on shrinkage results. Different-sized specimens have different surface-to-volume ratios, which will affect the rate of shrinkage (Hansen and Mattock 1966). A 1 in. (25 mm) square specimen shrinks more quickly than 3 or 4 in. (75 or 100 mm) square specimens. If larger specimens are used, it may be appropriate to consider shrinkage at 16 weeks instead of 28 days. Comparison of shrinkage results should always be made using the same specimen dimensions, curing regimen, and storage conditions.

3.2.2 ASTM C157 as modified by ICRI 320.3R—ICRI 320.3R describes a modification of ASTM C157 and makes recommendations for reporting properties appropriate for cement-based repair materials. The standard test specimen is 3 in. (75 mm) square by 11-3/4 in. (300 mm), so the same surface area-to-volume ratio exists for mortar, extended mortar, and concrete. Non-polymer-cement materials are cured in a moist room for 24 hours, or for 2 hours after final setting time for rapid-hardening materials, and then demolded. Initial comparator readings are made based on final setting time as an indicator of development of sufficient strength to handle the bar. Polymer-cement materials are covered with polyethylene film immediately after casting, and demolded after 24 hours, or 2 hours after final setting time for rapid-hardening materials, in accordance with ASTM C1439. Initial comparator readings are made. The specimens are stored in a drying room or a water tank, and comparator readings are made at 3, 7, 14, 28, and 56 days. Measurements continue until 90 percent of the ultimate drying shrinkage or moisture expansion, as determined by ASTM C596, is attained.

3.2.3 ASTM C596—Length change during drying is determined for flowable mortar containing hydraulic cement and graded standard sand. Test specimens are 1 in. (25 mm) square by approximately 11-1/4 in. (285 mm). Specimens are moist cured for 24 hours, demolded, and cured in lime-saturated water for 48 hours. Length comparator measurements follow. The specimens are then stored in air for 25 days, and length comparator measurements are made after 4, 11, 18, and 25 days of air storage. Typical shrinkage strains range from -0.05 to -0.15 percent. In general, shrinkage values more negative than -0.10 percent are considered too high for concrete repair (Vaysburd et al. 1999). Some repair materials have shrinkage values more negative than this criterion, and may be inappropriate for use in applications where shrinkage may be an issue.

3.2.4 ASTM C806—This test method is typically used for comparative evaluation of expansive cements using a prescriptive mortar mixture. It is inappropriate for use with repair mortars because the test specimens are cured under water continuously for 28 days while repair mortars are usually exposed to air drying within 1 week after placement. Test specimens are 2 in. (50 mm) square by 10 in. (250 mm) long. A restraining cage, including a steel rod, is cast with the specimen. Immediately after casting, the specimens are covered with polyethylene sheets. After 6 hours, the specimens are demolded. The specimens are then cured in lime-saturated water for 28 days. Comparator measurements are made after 7 and 28 days, and the length change is calculated. A typical value for expansion is 0.06 percent under the conditions of this test.

3.2.5 ASTM C827—The early-age movements of cylindrical specimens of cementitious mixtures of paste, grout, mortar, and concrete are measured. Depending on the maximum-size aggregates, specimen size varies from 2 in. (50 mm) diameter by 4 in. (100 mm) high to 6 in. (150 mm) diameter by 12 in. (300 mm) high. Immediately after molding the specimens, an indicator ball is partially embedded in the top of each specimen. The test specimen is placed between a projected light source and a magnifying lens system, and movements of the ball are enlarged optically and measured on a wall chart. Measurements are made at 5-minute intervals for the first 90 minutes, at 10-minute intervals for the next hour, and at 20-minute intervals thereafter until the mixture has hardened. The change in height of the specimen is then calculated.

3.2.6 ASTM C1581—This test measures the strains in a cylindrical specimen cast around a steel ring on a smooth, nonabsorbent, nonbonding surface, shown in Fig. 3.2.6. The specimens have an inside diameter of 13 in. (330 mm), an outside diameter of 16 in. (405 mm), and a height of 6 in. (150 mm). Unless otherwise specified, the specimens are wet cured for 24 hours and then demolded, leaving the instrumented inner steel ring in place. The top surface is then sealed with paraffin wax or adhesive aluminum-foil tape. As the repair material shrinks, strain is transferred to the steel ring, which is fitted with strain gauges. Strains are measured every 30 minutes maximum, and the specimens are visually inspected every 3 days maximum. The specimens are monitored for at least 28 days after initiation of drying, unless cracking occurs prior to 28 days. The age at cracking is indicated when the strain suddenly decreases.

The test is intended to give a relative indication of when cracking due to restrained shrinkage may occur for different repair materials. The results provide a comparative measure of a material's tendency to crack, which is affected by a combination of several material properties, including drying shrinkage, modulus of elasticity, tensile strength, strain capacity, and creep. Temperature changes and coarse aggregate effects are not considered.

3.3—Mechanical properties

Mechanical properties affect repair material interactions with the substrate concrete. It is essential that the repair

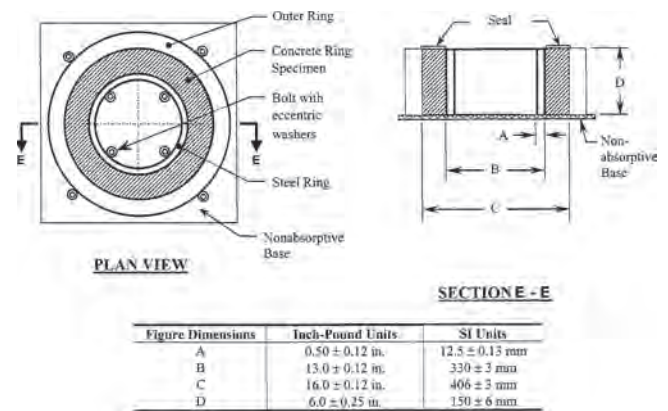


Fig. 3.2.6—Schematic of ASTM C1581 ring test specimen. (Reprinted with permission from ASTM C1581.)

material remain bonded to the concrete. It is also important that the repair material has mechanical properties compatible with those of the substrate concrete to ensure that the materials act as one and no material failure occurs. If some properties of the repair material are greatly different from those of the substrate concrete, such as the coefficient of thermal expansion, other properties should compensate for these differences for the repair to perform successfully. For example, a lower modulus of elasticity will reduce the stresses due to differing thermal movements. Usually it is not necessary, or even desirable, for the repair material to have some mechanical properties, such as compressive strength, tensile strength, or bond strength, substantially in excess of the substrate concrete, as any subsequent failures would simply occur in the weaker substrate concrete adjacent to the repair.

3.3.1 Elasticity—Elasticity is a material property that causes the material to recover its original size and shape after an applied deformation or force is removed. Elasticity is primarily important for materials intended to bridge active cracks, such as some crack sealants and surface coatings. Elasticity is typically quantified by measuring the elongation of a material in tension, as in ASTM D638, discussed in 5.3.2.

3.3.2 Static modulus of elasticity—If the repair is not structural (not intended to share load with the substrate concrete), then it is typically desirable that the repair material have a modulus of elasticity lower than that of the substrate concrete, so it can more readily accommodate future movements within the repair material and at the interface between the repair material and substrate concrete. A lower modulus is particularly desirable if the repair material has volume stability or thermal compatibility properties that differ significantly from those of the substrate concrete. If a repair is intended to share load with the existing structure (structural repair), it is desirable for the moduli of elasticity of both materials to match as closely as possible. If the repair material has a higher modulus of elasticity, it will attract more of the applied load; if the repair material has a lower modulus of elasticity, it will deform under stress and transfer load into the substrate. Significant deviations can lead to uneven load distribution and system failure.

Two ASTM test methods typically used to measure the modulus of elasticity of repair materials are:

- 1) **ASTM C469**
- 2) **ASTM C580**

In **ASTM C469**, a molded cylindrical specimen or a drilled core specimen is loaded in compression and the axial deformation measured. Load-deformation data is plotted and a chord modulus of elasticity is calculated for a stress corresponding to 40 percent of the ultimate load. The test setup is shown schematically in Fig. 3.3.2a. In **ASTM C580**, a square-bar beam specimen is loaded at midspan in flexure and the midspan deflection is measured. Load-deflection data are plotted and a secant modulus of elasticity is calculated for a deflection that is 50 percent of the maximum deflection. The test setup is shown schematically in Fig. 3.3.2b. The results reported in stress units (psi or MPa).

3.3.3 Coefficient of thermal expansion—In situations where temperature is not controlled, such as in exterior and some interior applications, it is desirable for the repair material to have a coefficient of thermal expansion similar to that of the substrate concrete, so the two materials behave similarly under daily and seasonal temperature variations. If the coefficients vary significantly, the differential movements due to temperature fluctuations could affect the perfor-

mance of the repair and should be accounted for in the repair design. The coefficient of thermal expansion for concrete typically ranges from 0.000002 to 0.000008/°F (0.000004 to 0.000014/°C), depending primarily on the aggregate type.

Four test methods are used to determine the coefficient of thermal expansion.

- 1) **ASTM C531**
- 2) **ASTM D696**
- 3) **ASTM C884**
- 4) United States Army Corps of Engineers (USACE) **CRD-C 39**

ASTM C531 and D696 and USACE CRD-C 39 are test methods in which the coefficient of thermal expansion is measured by determining the change in length of a material at constant humidity between two different temperatures: 73°F and 210°F (23°C and 100°C) in ASTM C531; and -22°F and +86°F (-30°C and +30°C) in ASTM D696. Results are reported as strain per unit temperature change. ASTM C884 is a test method where thermal compatibility between concrete and an epoxy resin overlay is determined by a similar approach. In this case, a concrete substrate with an epoxy-resin overlay is cycled through a temperature range between 77°F and -6°F (25°C and -21°C) five times. If the epoxy resin overlay debonds from the concrete or there are horizontal cracks in the concrete near the interface, the epoxy resin system fails the test.

3.3.4 Creep—Because many repairs are not subjected to significant compressive forces, compressive creep may not be a significant property of repair materials. Creep can be important in reducing stress induced in the repair material due to restraint of shrinkage strains or factors, such as thermal movement or the application of live loads.

Two ASTM test methods are used to evaluate compressive creep:

- 1) **ASTM C512**
- 2) **ASTM C1181**

Both tests are suitable for testing repair materials.

In ASTM C512, creep is determined by placing a material under a sustained stress, usually not more than 40 percent of the compressive strength at the age of loading, measuring the resultant strain over time, and reducing the strain by measured strains from unloaded control specimens, which

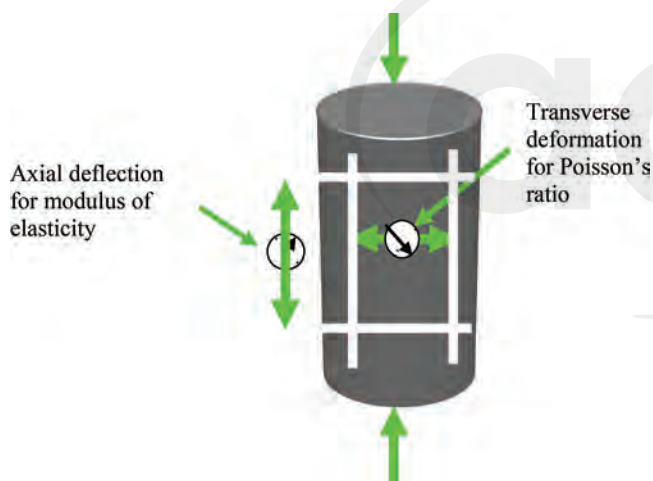


Fig. 3.3.2a—Schematic of ASTM C469 test setup. (Courtesy of BASF Construction Chemicals, LLC.)

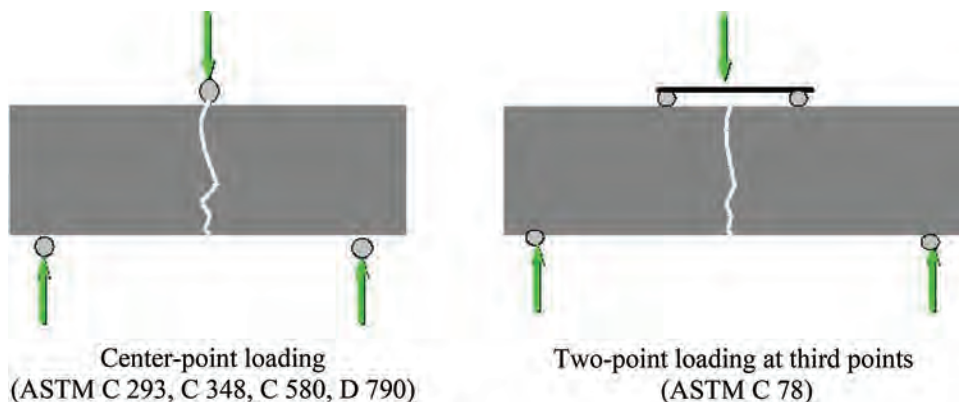


Fig. 3.3.2b—Schematics of flexural test setups. (Courtesy of BASF Construction Chemicals, LLC.)

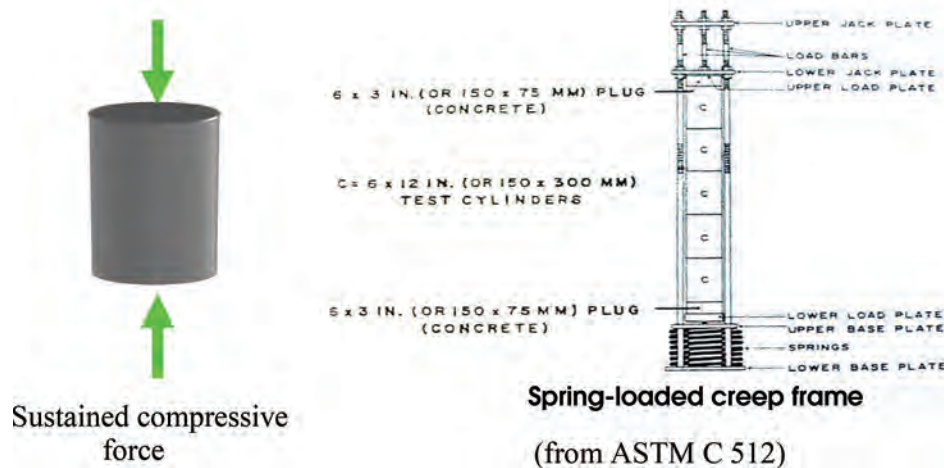


Fig. 3.3.4—Schematic of ASTM C512 creep loading. (Reprinted with permission from ASTM C512) and spring-loaded creep frame (Courtesy of BASF Construction Chemicals, LLC).

account for strains due to shrinkage. To determine the complete creep behavior of a repair concrete, test specimens are loaded at ages of 2, 7, 28, and 90 days, and 1 year. The creep strains are measured at specified intervals in the first year. Results are reported as percent strain at a given age for a given sustained stress. Creep may be determined for materials of different ages and under different curing regimens; however, variation from standard conditions should be explicitly reported. Figure 3.3.4 shows schematics of a typical creep test setup.

In [ASTM C1181](#), test specimens are conditioned for 7 days and are then tested for 35 days. A test stress is selected and applied to the specimen, and the specimen deflection is measured. The specimen is then unloaded and placed in an oven at a selected temperature for 24 hours, after which the specimen is allowed to cool at 73°F (23°C) for 24 hours. The specimen is then reloaded to the selected stress and the deflection measured. This heating, cooling, loading, and deflection measurement cycle is repeated five times, with subsequent heating periods of 24, 72, and 48 hours, and 7 and 28 days. Creep is expressed as a graph of strain versus time in the oven.

Tensile creep is a significant property for repair materials. Tensile creep can reduce the cracking of repair materials by relaxing the tensile stress over time. No commonly accepted, standardized test method is available for measuring tensile creep, and there is no consensus regarding a correlation between tensile and compressive creep ([Pigeon and Bissonnette 1999](#); [Iriya et al. 1999, 2000](#); [Beaudoin 1982](#)).

3.3.5 Bond strength—Bond strength, or adhesion, relates to the ability of the two materials to act as one. This section addresses bond between the repair material and substrate concrete only.

A repair material should have sufficient bond strength to the substrate concrete so it does not separate from the substrate concrete. It is desirable for the bond strength to be larger than the minimum requirements so that any reductions due to field-application conditions are not as critical. Bond

strengths that exceed the tensile strength of either the repair material or the substrate will induce failure in the weaker material if sufficient interface stresses result from shrinkage, thermal movement, or other factors.

Seven test methods are used to measure bond strength:

- 1) [ASTM C882](#)
- 2) [ASTM C1042](#) (withdrawn 2008)
- 3) [ASTM C1404](#) (withdrawn 2010)
- 4) [CSA A23.2-6B](#)
- 5) [ASTM C1583](#)
- 6) [ICRI 210.3](#)
- 7) [Michigan Department of Transportation \(MDOT\)](#)

Direct Shear Bonding Test

In these test methods, bond strength is usually inferred from the measured slant shear, direct tension, or direct shear test values, because failure rarely occurs at the bond line. Failures that occur away from the bond line imply that the bond strength is greater than the failure load in the test. When failure occurs at the bond line in direct tension tests, the measured tensile force is the actual bond strength. While ASTM C1042 and C1404 have been withdrawn, some material manufacturers still report bond strengths based on these procedures.

Slant shear bond tests (ASTM C882 and C1042) measure the resistance to sliding between the repair material and concrete substrate along an inclined surface. The interface is subjected to combined shear and compressive stress. The measured bond strength may be influenced by the compressive strength of the substrate concrete and the surface roughness of the bonding surface. Direct tension bond tests (ASTM C1404, CSA A23.2-6B, ASTM C1583, and ICRI 210.3) are more appropriate for repair applications because they apply a single mode of stress and direct tension at the bond interface. The ASTM C1404 test is a laboratory test for adhesive systems, while the CSA A23.2-6B, ASTM C1583, and ICRI 210.3 tests use a cored specimen drilled through the repair material into the substrate. The latter three tests are suitable for field and laboratory evaluation. [ASTM C1583](#) and [ICRI](#)

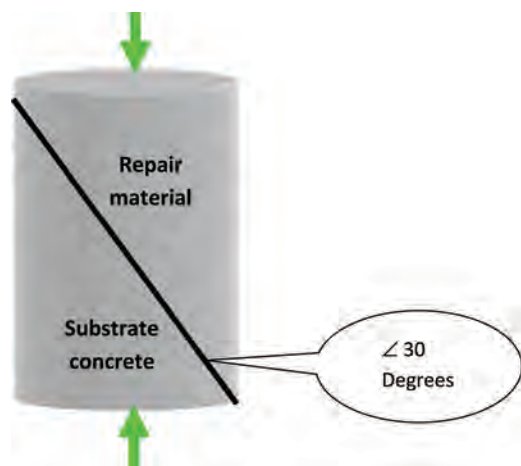


Fig. 3.3.5a—Schematic of ASTM C882 and C1042 slant shear tests. (Courtesy of BASF Construction Chemicals, LLC.)

210.3 test procedures were introduced in 2004 and have had limited time for industry use. The MDOT direct shear bonding test measures the shear applied along the interface between the repair material and sawn substrate concrete.

ASTM C882 and **C1042** are slant shear bond tests specifically for epoxy-resin and latex bonding systems, and are the most commonly reported test results. These tests involve casting a 3 x 6 in. (75 x 150 mm) right cylinder of substrate mortar or concrete using a dummy section to form a planar bonding surface that is 30 degrees off the longitudinal axis of the cylinder. The bonding surface is prepared by sandblasting or diamond sawcut (ASTM C1042 only), the bonding material is applied, and the test material is cast on top of the test specimen to complete the 3 x 6 in. (75 x 150 mm) right cylinder. At the appropriate age, the test specimen is loaded to failure in compression. The stress at fracture is determined by dividing the ultimate load by the elliptical area of the bonding surface to determine the bond strength. If the specimen does not fail at the bond line, the bond strength is considered to be at least the resulting failure stress. The substrate strength, porosity, surface roughness, moisture content, and other factors should be considered in evaluating values determined from these tests. Tests are shown schematically in Fig. 3.3.5a.

ASTM C1404 is a direct tension test procedure that involves cutting a 3 x 6 in. (75 x 150 mm) concrete cylinder in half, creating a planar bonding surface perpendicular to the longitudinal axis of the cylinder. The cut surface is rubbed with abrasive paper, the adhesive being tested is applied to the cut surface, and a mortar overlay is applied, completing the 3 x 6 in. (75 x 150 mm) right cylinder. Steel pipe nipples are attached to the specimen, the nipples are screwed into pipe caps, the pipe caps are attached to the test machine, and the specimen is loaded to failure in direct tension. Stress at fracture is determined by dividing the ultimate load by the cross-sectional area of the cylinder. If the specimen does not fail at the bond line, bond strength is at least the stress at fracture.

CSA A23.2-6B, **ASTM C1583**, and **ICRI 210.3** are test procedures that involve coring through a concrete replace-

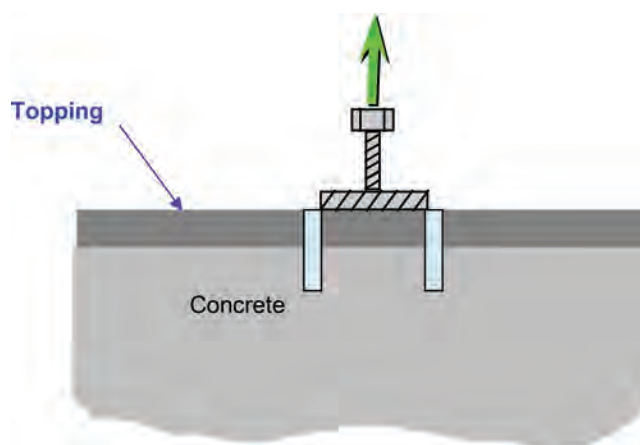


Fig. 3.3.5b—Schematic of direct tension bond test setup (CSA A23.2-6B). (Courtesy of BASF Construction Chemicals, LLC.)

ment or overlay material into the substrate concrete; adhering with epoxy a rigid plate or a machined, standard pipe cap with a minimum outside diameter of 2 in. (50 mm) to the core surface; and pulling along the core axis to failure at a rate of approximately 15 to 23 lb (70 to 100 N) per second. The procedure is shown schematically in Fig. 3.3.5b. The failure type and load are recorded, and the failure load is reported in terms of psi (MPa).

Direct tension values below 100 psi (0.69 MPa) most likely indicate a serious problem with the repair material bond. As the direct tension bond strength approaches 200 psi (1.4 MPa), the bond interface strength begins to approach the tensile strength of the concrete substrate, which is a limiting value of the overall repair (Knab et al. 1989).

The **MDOT Direct Shear Bonding Test** is a direct shear test that is performed with a 4 in. (100 mm) cube or a cast or cored cylinder. For cube specimens, substrate concrete is cast into a cube and a 1 in. (25 mm) thick section is sawn from one face. The resulting surface is cleaned and verified to have a uniform profile, and the test material is cast onto the substrate to reform a 4 in. (100 mm) cube. The cube is then secured in a test machine and direct shear force is applied to the bonding surface. Stress at failure is determined by dividing the ultimate load by the area of bonding surface. A similar procedure is used with a right cylinder. A cored cylinder can be used to test in-place materials. Direct shear bond strengths of 300 to 500 psi (2.1 to 3.4 MPa) are generally considered to be adequate (Knab et al. 1989). The test setup is shown in Fig. 3.3.5c.

3.3.6 Compressive strength—Three test methods are used to measure compressive strength:

- 1) **ASTM C39**
- 2) **ASTM C109**
- 3) **ASTM C579**

Compressive strength is determined by applying an increasing axial compression load until the specimen is unable to support additional load. The ultimate load is divided by the cross-sectional area to determine the ultimate failure stress (psi or MPa). **ASTM C39** tests cylindrical

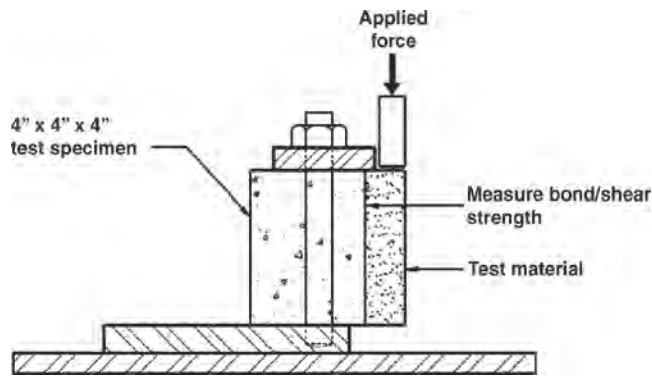


Fig. 3.3.5c—MDOT direct shear bonding test setup. (Courtesy of Wiss, Janney, Elstner Associates, Inc.)

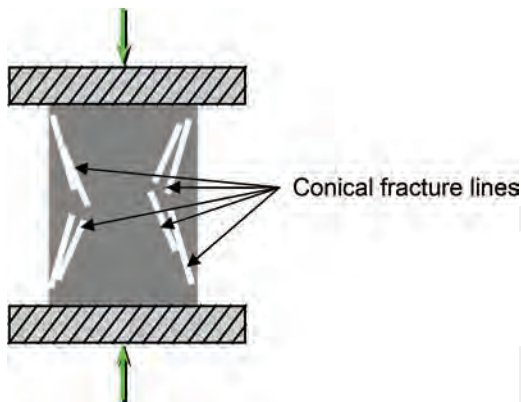


Fig. 3.3.6—Schematic of compression test setup (ASTM C39; C109; C579). (Courtesy of BASF Construction Chemicals, LLC.)

concrete specimens that are 4 x 8 in. (100 x 200 mm) or 6 x 12 in. (150 x 300 mm). ASTM C109 tests 2 in. (50 mm) mortar cubes. ASTM C579 tests right cylinders 1 x 1 in. (25 x 25 mm) for materials with aggregates less than 1/16 in. (1.6 mm) in size; 2 in. (50 mm) cubes for materials with aggregates from 1/16 to 0.4 in. (1.6 to 10 mm) in size; and 2 in. (50 mm) minimum diameter right cylinders, with a height twice the diameter, for materials with aggregates larger than 0.4 in. (10 mm). A schematic of the test setup is shown in Fig. 3.3.6. Usually, it is desirable to have a compressive strength similar to that of the substrate concrete, but at least 4000 psi (28 MPa). The fracture pattern of the specimens should also be documented according to [ASTM C39](#).

3.3.7 Tensile strength—Three test methods are used to measure tensile strength:

- 1) [ASTM C307](#)
- 2) [ASTM C496](#)
- 3) [USACE CRD-C 164](#)

Tensile strength of cementitious materials is usually determined by a splitting tensile test (ASTM C307 and C496). The splitting tensile test applies a compressive force along the axis of an unconfined cylinder supported on its side, causing it to split along its axis, as shown schematically in Fig. 3.3.7a. Results are reported in stress (psi or MPa) and are normally of greater magnitude than direct tensile



Fig. 3.3.7a—Schematic of splitting tension test setup (ASTM C307; C496). (Courtesy of BASF Construction Chemicals, LLC.)



Fig. 3.3.7b—Schematic of direct tension test setup (USACE CRD-C 164). (Courtesy of BASF Construction Chemicals, LLC.)

strength measurements (USACE CRD-C 164). Direct tensile strength measurements provide a true indication of the tensile strength of the material because the tensile stress is applied perpendicular to the failure plane. The direct tension test setup is shown schematically in Fig. 3.3.7b.

3.3.8 Flexural strength and modulus of rupture—Five test methods are used to measure the flexural strength of different repair materials:

- 1) [ASTM C78](#)
- 2) [ASTM C293](#)
- 3) [ASTM C348](#)
- 4) [ASTM C580](#)
- 5) [ASTM D790](#)

Flexural strength is determined by casting a beam specimen of a given material and testing it in bending. Tests use concrete beams supported at their ends with either one (ASTM C293, C348, C580, and D790) or two (ASTM C78) loading points, as shown schematically in Fig. 3.3.2b. The one-point loading at midspan creates the maximum tensile stress at a single cross section where the load is applied, whereas the two-point loading at the third points produces the maximum tensile stress over the middle third of the specimen, making the test less susceptible to scatter due to non-uniform material or the presence of large aggregate at the loading point. Tests have shown that the flexural strength from third-point loading could be approximately 75 psi (0.52 MPa) less than that obtained from center-point loading ([Portland Cement Association 1995](#)). The resulting flexural strength is reported in stress units (psi [MPa]).

3.3.9 Significance of compressive, tensile, and flexural strengths—Compressive and flexural strengths are normally not the limiting properties in the performance of repair materials but may be used as general indicators of the material quality. Usually it is desirable for these properties to meet or slightly exceed those of the substrate concrete (as measured at the time of the repair, not as originally specified). Higher tensile strength could be beneficial in reducing cracking due to restrained shrinkage and thermal contraction.

3.4—Constructibility characteristics

Constructibility characteristics are those material properties that affect or limit the installation or application of the repair material for varying field conditions, such as overhead or formed and poured repairs and hot or cold weather work.

3.4.1 Cohesiveness—The cohesiveness of a repair material is its ability to remain intact or not segregate during its application. Cohesiveness of repair material is important for the ease of construction and uniformity of repair. For example, in concrete replacements on vertical and overhead surfaces, a more cohesive material can be applied in thicker lifts with less chance of internal separations or debonding before setting.

3.4.2 Viscosity—The measured value of viscosity is affected by factors such as the method of measurement (viscometer or rheometer geometry), temperature, and the rate of applied shear strain. Materials with low viscosity flow more freely than those with higher viscosity. In general, low-viscosity materials are used to repair cracks and penetrate into concrete pores. Flowable, self-consolidating materials are also available for concrete replacements.

3.4.3 Working time—Factors such as ambient and material temperature, mixing ratio, and mass of material can significantly affect the amount of time available for the applicator to spread the material. Consequently, it is difficult to test for and report working time in a meaningful manner. A material may be unworkable before initial set occurs, so that working time is less than or equal to initial set time.

3.4.4 Environmental considerations—Repair material should be suitable for the specific repair application environment. For example, some environmental limitations at the time of construction include the air and concrete temperatures, amount of moisture on the substrate concrete surface, relative humidity, wind speed, if the repair area is in direct sunlight or shade, and anticipated climatic conditions that occur before the repair material reaches its final set. The choice of specific repair materials and their corresponding properties depends on actual construction conditions. In some cases, the construction conditions are modified to fit the properties of the repair materials chosen.

3.5—Aesthetic properties

The appearance of repair materials is important in some circumstances.

3.5.1 Surface texture—Generally should match that of the adjacent material.

3.5.2 Color—Generally should match that of the adjacent material, unless the repair is not visible, such as after the application of a coating.

3.5.3 Aging—Some repair materials will change appearance as they age due to weathering, exposure to ultraviolet light, drying, or curing.

3.5.4 Moisture absorption—The appearance of repair material may change when it gets wet, and it may return to its original appearance as it dries. Changes in appearance may be different from the adjacent concrete.

3.6—Factors affecting durability

Service conditions can place various demands on the repair material, possibly requiring the repair material to have enhanced properties for long-term durability. Conditions can include exposure to moisture, temperature variations, chemicals, and mechanical wear.

3.6.1 Resistance to freezing and thawing—Some repair materials, including concrete, are susceptible to deterioration when exposed to cycles of freezing and thawing in a saturated condition. The expansion and migration of water can generate destructive internal pressures unless measures such as air entrainment are taken to provide the needed durability.

Three test methods are used to evaluate resistance to cyclic freezing and thawing:

- 1) **ASTM C666**
- 2) **ASTM C672**
- 3) **ASTM C672 as modified by ICRI 320.2R**

Durability of material in freezing environments is evaluated in bulk by ASTM C666, or as a surface scaling effect by ASTM C672 and modified by ICRI 320.2R. The approach in ASTM C666 is to expose concrete or a cementitious material to freezing-and-thawing cycles and record the change in dynamic modulus of elasticity and mass loss of the specimens. ASTM C672 relies on a visual inspection of surface damage caused by freezing-and-thawing cycles of a ponded salt solution, while the ICRI 320.2R modification includes measuring the mass loss of the scaled material to quantify the amount of scaling.

ASTM C666—This test method includes two procedures. Procedure A is with both freezing and thawing in water, and Procedure B is with freezing in air and thawing in water. Procedure A is the most commonly used because it is more easily automated. A complete test consists of 300 freezing-and-thawing cycles, unless the relative dynamic modulus of elasticity of the specimen first reaches 60 percent of the initial value. A durability factor, as calculated in the test procedure, which is greater than 80, is generally considered to represent a material that will be durable under freezing-and-thawing conditions. If this test is not carried out to 300 cycles, the total number of cycles should be reported. ASTM C666 is considered a severe test method; some materials that fail the test may not fail under harsh service conditions.

ASTM C672—This test method, commonly called the salt scaling test, is appropriate for materials exposed to freezing-and-thawing cycles in conjunction with deicing salt exposure. A 4 percent solution of calcium chloride is applied to the specimen surface and subjected to cyclic freezing and

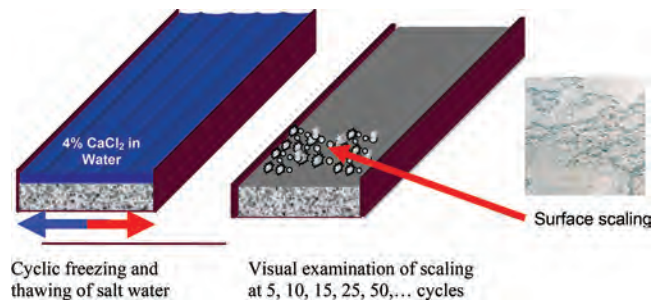


Fig. 3.6.1—Schematic of ASTM C672 freezing-and-thawing test setup. (Courtesy of BASF Construction Chemicals, LLC.)

thawing. Each cycle lasts 24 hours and, generally, 50 cycles are sufficient to evaluate a surface. Test results use a subjective visual scale of 0 to 5, with 0 indicating no scaling and 5 indicating severe scaling. Refer to Fig. 3.6.1 for a schematic of the test setup.

ICRI 320.2R provides a modification of ASTM C672 that is often used to eliminate subjectivity of the standard test. The modification involves collecting, drying, and weighing the scaled material to quantify the mass loss from surface deterioration after every 10 cycles. A value less than 0.16 lb/ft² (0.8 kg/m²), after 50 cycles, usually denotes acceptable performance.

As durability of the concrete surface improves with age, the age of test specimens when initially exposed to the freezing-and-thawing cycling is critical to interpreting the results. ASTM C672, with or without the ICRI 320.2R modification, has been found to be a severe test method; some materials that fail the test may not fail under harsh service conditions. ASTM C672, with or without the ICRI 320.2R modification, has also been found to be sensitive to surface finishing techniques (Johnston 1993). Conducting the test on a formed versus finished face of a specimen can significantly affect the results.

3.6.2 Permeability—The permeability of a repair material is important in environments where the repair material or substrate concrete is vulnerable to moisture-related deterioration such as freezing-and-thawing damage of saturated concrete, corrosion of embedded reinforcing steel, alkali-aggregate reactions, or sulfate attack. Permeability is particularly sensitive to the age of the material being tested. Generally, permeability decreases as the cementitious material hydrates or as the level of carbonation increases. Tests should always indicate the specimen age at the time of the test.

Five test methods are used to measure permeability:

- 1) American Association of State and Highway Transportation Officials (AASHTO) T259
- 2) ASTM C1543
- 3) AASHTO T277
- 4) ASTM C1202
- 5) ASTM C642

The permeability of concrete or mortar is sometimes determined by its resistance to chloride ion penetration. AASHTO T259 consists of ponding a sodium chloride solution on the top surface of slab specimens for 90 days and measuring the chloride-ion content at different levels within the concrete.

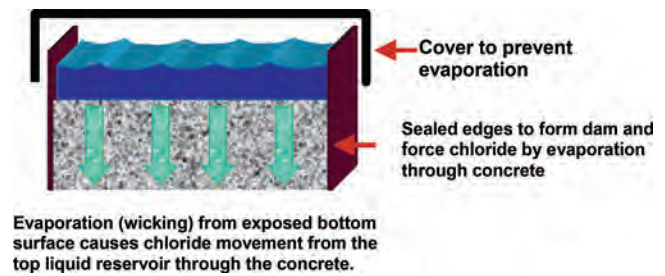


Fig. 3.6.2a—Schematic of AASHTO T259 and ASTM C1543 test setup. (Courtesy of BASF Construction Chemicals, LLC.)

The results can be used to compare different concretes and repair materials. ASTM C1543 is similar to AASHTO T259, except that it does not have a specific test period. AASHTO T277 and ASTM C1202 were developed to provide an indication of a material's resistance to chloride-ion ingress in a shorter period of time, which is typically 6 hours. In this procedure, a constant voltage is applied across a cylindrical specimen with ionic solutions on both sides of the specimen, and the total electrical current measured. The test does not actually measure the chloride-ion penetration but rather the electrical resistance of the immersed sample. As a result, the test results are influenced by the electrical resistivity of the test material and its permeability. ASTM C642 is a quick and economical method to approximate the permeable void content of concrete.

AASHTO T259 and ASTM C1543 consist of a 3 percent sodium chloride solution ponded on top of a concrete slab specimen for 90 days or longer. After 90 days, the concrete is sampled and analyzed to determine the chloride-ion content at various levels below the surface. In ASTM C1543, the testing entity may select a test period longer than 90 days. The findings are reported in percent chloride ions relative to the concrete or cement mass in the specimen, based on acid-soluble chloride. A schematic of the test setup is shown in Fig. 3.6.2a. AASHTO T259 requires 90 days to perform and ASTM C1543 usually requires at least 90 days to perform; the results can be used to compare different concretes and repair materials. Care should be taken to ensure that comparative tests are performed in the same manner; in particular, the test duration should be the same, the concrete should be sampled in the same manner for chloride testing, and the same chloride-ion test procedure should be used for all of the samples.

In the AASHTO T277 and ASTM C1202 procedure a cylindrical test specimen is placed in the test apparatus with a sodium chloride solution on one side and a sodium hydroxide solution on the other side, as shown in Fig. 3.6.2b(a). A constant voltage of 60 V is applied across the test specimen for 6 hours and the total electrical current passed is measured. In-progress tests are shown in Fig. 3.6.2b(b). The total charge passed, measured in coulombs (C), is taken as the measure of the permeability of the material. Generally, moderate-water-to-cementitious-material-ratio (w/cm) concrete (0.4 to 0.5) has values of 2000 to 4000 C; low-w/cm concrete (less than 0.4) has values ranging from 1000 to

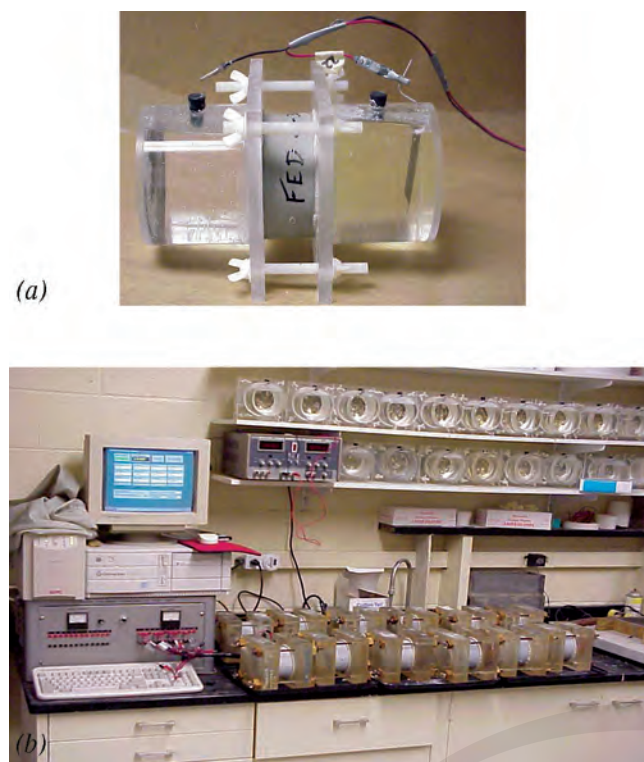


Fig. 3.6.2b—AASHTO T277 and ASTM C1202 rapid chloride permeability test: (a) test specimen; and (b) test in progress. (Courtesy of BASF Construction Chemicals, LLC.)

2000 C, and silica fume and polymer-cement concrete have values ranging from 100 to 1000 C (Whiting 1981).

Values obtained from AASHTO T277 and ASTM C1202 give an indication of a cementitious material's ability to resist the penetration of chloride ions, but the measured values should be interpreted with caution. Because the electrical current is measured, the electrical resistivity and permeability of the specimen will influence test results. To accurately predict chloride-ion permeability, the results of this test procedure should be correlated with the results of the long-term ponding test procedure described in AASHTO T259 or ASTM C1543. These tests are not suitable for testing steel fiber-reinforced concrete or other materials that influence the electrical conductivity of the material. Although the results are reported quantitatively, test values for properly conducted tests on the same material by different laboratories may differ by as much as 51 percent (ASTM C1202).

ASTM C642 is used to determine absorption after immersion, absorption after immersion and boiling, dry bulk density, bulk density after immersion, bulk density after immersion and boiling, apparent density, and volume of permeable pore space or voids. Boiling may result in increased values of absorption and bulk density. This method can provide an indirect indication of the permeability of cementitious paste repair materials. Permeable pore space data from this method should be supplemented by the other methods discussed in this section to provide a true indication of permeability. The volume of the permeable void result is affected by the material's porosity and damage to applied materials, such as cracking.

3.6.3 Alkali-aggregate reactions—Some aggregates are susceptible to deleterious expansive reaction with other components of the repair material, particularly with alkali in portland cement or chemicals introduced by the environment. These aggregates can be identified based on previous service records or with one or more of the test methods described herein. It is preferable not to use reactive aggregates in environments where deleterious reactions could occur. If they are used, steps should be taken to avoid the deleterious reactions, such as modifying the repair material mixture composition or protecting the material from the environment. Protection from the environment can be provided by water-repellent and waterproofing coatings. Relative humidity greater than 85 percent in the concrete has been shown to promote alkali-aggregate reactions (Stark 1991).

The five test methods used to evaluate alkali-aggregate reactivity potential are:

- 1) ASTM C227
- 2) ASTM C1260
- 3) ASTM C1293
- 4) ASTM C289
- 5) ASTM C295

ASTM C227, C1260, and C1293 all test the potential for alkali reactivity between a particular aggregate and cement by storing samples of the combined materials in prescribed conditions and monitoring the change in length over time. In ASTM C227, the specimens are conditioned for 1 day, then supported over water in a sealed container at 100°F (38°C) for 12 days, and then stored at 73°F (23°C) for 1 day. In ASTM C1260, the specimens are conditioned for 2 days and then stored in a 1 N solution of sodium chloride for 14 days. The ASTM C1293 procedure is similar to that of ASTM C227, except that ASTM C1293 is intended to assess the effects of a pozzolan or slag in the specimen and the duration of the test is 12 or 24 months. Materials with a length change of 0.05 percent or less are considered to have a high resistance to alkali-silica attack. ASTM C289 evaluates the reactivity of a particular aggregate by soaking the aggregate for 24 hours in a 1 N sodium hydroxide solution at 176°F (80°C). This method may not reliably predict the reactivity in concrete because the aggregates are tested in an alkali solution that is much different than in a concrete environment, and should only be used for initial screening. ASTM C295 describes a petrographic procedure to characterize aggregates, identify aggregate types that are susceptible to alkali-aggregate reaction, and identify other deleterious properties of aggregates that can harm the concrete. Additional discussion is found in ACI 221.1R and CSA A864.

3.6.4 Electrical resistivity—Corrosion is an electrochemical process. Therefore, if corrosion of embedded reinforcing steel is a concern, the electrical resistivity of the repair material is important, particularly for concrete replacements. Depending on the method of corrosion protection, a high or low value of this property may be desirable. Cathodic protection systems, both impressed current and galvanic, generally require a lower value (approximately 300 ohm-cm) than higher resistance value (1000 to 5,000 ohm-cm) desired in

conventional repairs ([ASTM C1202](#)). Refer to [ACI 222R](#) for further guidance. One indication of the resistivity of a repair material can be obtained from [AASHTO T277](#) or [ASTM C1202](#), as discussed in [3.6.2](#).

3.6.5 Abrasion resistance—Many repairs are subjected to abrasion in service. The abrasion can be from wind-blown debris, mechanical contact, or other sources. In particular, vehicular traffic and abrasion on hydraulic structures are aggressive types of abrasion. Repair materials should have suitable abrasion resistance for their service environment.

Four test methods are used to measure abrasion resistance:

- 1) [ASTM C779](#)
- 2) [ASTM C944](#)
- 3) [ASTM C418](#)
- 4) [ASTM C1138M](#)

These procedures measure surface resistance to different types of abrasive actions. The tests are intended to determine variations in the surface properties of concrete affected by mixture proportions, finishing, and surface treatment. They provide relative results between concretes but are not intended to provide a quantitative measure of the service life that may be expected from a specific surface.

[ASTM C779](#) includes three procedures for determining the relative abrasion resistance of horizontal concrete surfaces based on depth of wear measurements. The three procedures differ in the type and degree of abrasive force they impart. Procedure A uses revolving disks with No. 60 silicon carbide grit that slide and scuff the concrete surface in a circular path. Procedure B uses dressing wheels riding in a circular path to abrade the concrete surface by impact and sliding friction. Procedure C uses rapidly rotating ball bearings in a race and under load to impart high-contact stresses, impact, and sliding friction to a concrete surface. Procedures A and B are run for 30 minutes when depth-of-wear readings are taken. Procedure C is run for 20 minutes or until a maximum wear depth of 0.12 in. (3.0 mm) is reached, with depth of abrasion measurements taken at least every 50 seconds. A test-in-progress is shown in [Fig. 3.6.5](#). [ASTM C944](#) is a test procedure where rotating cutters are applied to the concrete surface under a normal load of 22 or 44 lb (98 or 197 N) for three 2-minute periods. After each period, the weight loss of the specimen is determined.

[ASTM C418](#) determines the abrasion-resistance characteristics of a concrete surface when subjected to the impingement of air-driven silica sand using specified pressures, abrasive, and sandblasting equipment. Wear is determined volumetrically by filling the eroded cavity with a weighed quantity of putty. Comparative data between samples make it possible to rank abrasion resistance.

[ASTM C1138M](#) determines the relative resistance of concrete to underwater abrasion by simulating the abrasive action of waterborne particles such as silt, sand, gravel, and other solids. A 12 in. (300 mm) diameter by 4 in. (100 mm) tall test specimen is immersed 6-1/2 in. (163 mm) under water in a steel pipe. A rotating mixing paddle propels 70 Grade 1000 chrome-steel grinding balls, graded from 1/2 to 1 in. (13 to 25 mm) in diameter, at the sample surface. The



Fig. 3.6.5—Measuring depth of wear during ASTM C779 abrasion test. Note disks (Procedure A) and dressing wheels (Procedure B) at top of machine. (Courtesy of CTL Group.)

loss in mass is measured every 12 hours for a test period of 72 hours.

3.6.6 Resistance to chemical attack—Repair materials may be exposed to various chemicals in the service environment. The long-term performance of the repair may depend on the proper selection of repair material and possibly on the use of an appropriate protective barrier. In many environments, it may be important that the barrier be free of pinholes and holidays. Some repair materials may be damaged by a solvent-based barrier material. Detailed discussions of resistance to chemical attack are found in [ACI 201.2R](#) and [PCA IS001](#) ([Portland Cement Association 2007](#)). In addition, a number of materials and solutions that are aggressive to concrete are listed in [PCA IS001](#).

Aggressive chemicals can be loosely categorized as sulfates, inorganic and organic acids, alkaline solutions, salt solutions, solvents, and others.

Portland-cement mortar and concrete could experience a destructive expansive reaction when exposed to sulfates in groundwater or from other sources. [ASTM C1012](#) evaluates the susceptibility of a cement mortar to sulfate attack by measuring the length change of a specimen over a period of 12 or 18 months while immersed in a sodium sulfate solution. The length comparator setup is shown in [Fig. 3.2.1](#). Results are presented as percent length change at a given time of exposure. A material with a length change of 0.05 percent or less is considered to have high resistance to sulfate attack.

Acids are those materials with a pH below 7. They react with concrete and cement-based repair materials to form compounds that are water-soluble. Acids could also attack

carbonate aggregates (limestone and dolomite). The effect of an acid is influenced by its concentration and chemical type, length of exposure, and temperature.

Alkaline solutions are those materials with a pH above 7. They are less aggressive than acids to portland-cement-based materials, but may react with other types of repair materials, depending on concentration, chemical type, temperature, and the chemical composition of the repair material. Hydrated cement is alkaline, so most repair materials have at least some resistance to high-pH environments. Strong concentrations of sodium or potassium alkalis can induce alkali-silica reactions if susceptible aggregates are present.

Salt solutions can be slightly acidic or alkaline and, depending on the chemical composition, might attack concrete and concrete repair materials. Salt scaling is one form of chemical attack (Taylor 1990).

Several solvents are aggressive to polymeric materials and others slowly attack concrete. A more common problem is previous solvent contamination of the substrate concrete, which can adversely affect the bond of the repair material.

Other chemicals are aggressive to concrete and repair materials. Extremely soft or purified water could slowly leach the calcium from cement-based concrete and mortar.

3.7—Chemical composition

3.7.1 Repair material chemistry—Repair products are sometimes independently tested for comparison with competitive products, for product approval purposes, or to ensure the chemical formulation is not significantly changed.

3.8—Summary tables

As concrete mixtures may include different types of portland cement, different gradations and types of fine and coarse aggregates, and different mixture proportions, the replacement concrete properties can vary significantly. Table 3.8a presents a brief summary of the test procedures described in detail in this chapter, including typical test values for basic portland-cement replacement concrete.

Where recommended test values are given in Table 3.8a, they represent the minimum values recommended by Committee 546 for typical repair materials used in successful repairs and are not necessarily applicable for all conditions. For most of the tests, Committee 546 has indicated if the test is recommended for evaluating repair materials. Some recommended test methods are not appropriate for certain repair materials or repair applications. Some are useful for comparing different materials.

Basic portland-cement replacement concrete may be modified in a variety of ways:

- a) Substituting various hydraulic cements;
- b) Including chemical admixtures;
- c) Including mineral admixtures such as fly ash, silica fume, and natural pozzolans;
- d) Including polymer modifiers;
- e) Including or substituting slag cement;
- f) Including fiber reinforcement;
- g) Substituting a polymer for portland cement;

h) Omitting the coarse aggregate to create various replacement mortars.

There are an unlimited number of variations for basic portland-cement concrete. In general, the same test procedures are used to evaluate most replacement and overlay materials. Table 3.8b lists principle material properties that may change when additives and substitutions are made to basic portland-cement concrete. Although some typical values are presented, numerical values vary greatly and only trends in material properties are presented. For additional information, refer to ACI 225R for types of hydraulic cements, the influence of admixtures, and the influence of cement on properties of concrete; ACI 212.3R for chemical admixtures; ACI 232.2R for fly ash; ACI 234R for silica fume; ACI 232.1R for natural pozzolans; ACI 548.3R for polymer-modified concrete; ACI 233R for slag cement; ACI 544.1R for fiber reinforcement; and ACI 548.1R for polymer concrete.

CHAPTER 4—CONCRETE REPLACEMENT AND OVERLAY MATERIAL SELECTION

Every repair application places different demands on repair materials. Repair materials, therefore, should have suitable properties to meet the demands throughout the intended life of the repair. All repair materials have limitations, and the material specifier and user should select the materials with the highest likelihood of good, long-term performance. Table 3.8a provides a summary of the test methods available for comparing the properties of replacement and overlay materials. Material cost is often the deciding factor among comparable materials. While life-cycle cost is a better comparator between alternate repair materials, life-cycle costs are often unavailable or approximate at best, leaving the repair specifier with a choice based on the initial cost of the materials and their application for materials with anticipated similar durability.

Many of the proprietary products for concrete repair that are available commercially are blends of several types of materials. Because these materials are proprietary, a list of their ingredients is usually unavailable to the specifier, except possibly in terms of generic materials. Prior experience with specific products and the results of independent testing of products are useful in evaluating such products. In this regard, many State and Canadian Provincial Departments of Transportation (DOT) maintain lists of approved proprietary concrete repair materials. These lists provide useful input to the specifier and user of products that have been found satisfactory by those agencies. Because repair products may be reformulated and their physical properties altered over time, some DOTs, such as the Alberta Transportation, Canada, periodically update their approval lists based on regular retesting. The American Association of State Highway and Transportation Officials (AASHTO) maintains a Product Evaluation List Management System, APEL, (<http://www.ntpep.org/Pages/APEL.aspx>) (accessed March 28, 2014), where the findings of evaluations of new, proprietary, or both new and proprietary engineering products are voluntarily posted.

Table 3.8a—Summary of available test methods and test values for portland-cement replacement concrete

Description	Test method	Typical value	Recommended value*	Recommended test
3.2—Volume stability				
Length change – concrete†	ASTM C157	0.02 percent (expansion) to –0.05 percent (shrinkage)	Less negative than (–0.05 percent) (shrinkage)	No; curing and comparator reading regimens not representative of field conditions of repair mortars and concretes.
Drying shrinkage – mortar†	ASTM C596	0.05 to 0.15 percent	< 0.10 percent	Yes
Restrained expansion	ASTM C806	0.06 percent	Refer to 3.2	Yes
3.3.2—Modulus of elasticity	ASTM C469	1,000,000 to 5,500,000 psi (6.8 to 38 GPa)	Refer to 3.3.3	Yes
	ASTM C580	300,000 to 3,000,000 psi (2.1 to 21 GPa)		Yes
3.3.3—Thermal expansion†	ASTM C531	0.000014/°F (0.000025/°C)	Refer to 3.3.3	Yes
	ASTM D696	0.000014/°F (0.000025/°C)		Yes
	USACE CRD-C 39	0.000006/°F (0.0000108/°C)		Yes
	ASTM C884	Qualitative test		Yes
3.3.4—Creep	ASTM C512	0.000000001/psi (0.000000007/KPa)	Refer to 3.3.4	Yes
	ASTM C1181	—		Yes
3.3.5—Bond strength				
Slant shear bond	ASTM C882	1 day – 400 to 1000 psi (2.8 to 6.9 MPa)	Committee 546 does not have a recommended value for this test.	No; results are highly dependent on compressive strength of substrate and roughness of bonding surface.
	ASTM C1042	7 days – 1000 to 1800 psi (6.9 to 12 MPa) 28 days – 2000 to 3000 psi (14 to 21 MPa)		Yes
Direct tensile bond	ASTM C1404	1 day – 70 to 150 psi (0.48 to 1.0 MPa)	Refer to 3.3.5	Yes
	CSA A23.2-6B	7 days – 150 to 250 psi (1.0 to 1.7 MPa)		Yes
	ASTM C1583	28 days – 250 to 300 psi (1.7 to 2.1 MPa)		Yes
	ICRI 210.3			Yes
Direct shear bond	MDOT	1 day – 150 to 300 psi (1.0 to 2.1 MPa) 7 days – 300 to 400 psi (2.1 to 2.8 MPa) 28 days – 400 to 600 psi (2.8 to 4.1 MPa)	Committee 546 does not have a recommended value for this test.	No, test apparatus not commonly available and test not commonly performed.
3.3.6—Compressive strength†	ASTM C39	28 days – 3000 to 10,000 psi (21 to 70 MPa)	Similar to substrate	Yes
	ASTM C109	28 days – 4000 to 12,000 psi (28 to 85 MPa)		Yes
3.3.7—Tensile strength†	ASTM C307	—	—	Yes
	ASTM C496	400 to 1800 psi (2.8 to 12 MPa)	> 400 psi (2.8 MPa)	Yes
	USACE CRD-C 164	200 to 600 psi (1.4 to 4.1 MPa)	> 400 psi (2.8 MPa)	Committee 546 does not have a recommendation for this test.
3.3.8—Flexural strength†	ASTM C78	28 days – 1200 psi (8.3 MPa)	Committee 546 does not have a recommended value for this test.	Yes
	ASTM C293	28 days – 500 to 1200 psi (3.4 to 8.3 MPa)		Yes
	ASTM C348	28 days – 1500 psi (10 MPa)		Yes
	ASTM C580	7 days – 2400 psi (17 MPa)		Yes
3.6.1—Resistance to freezing and thawing†	ASTM C666	28 days – 80 to 100 DF at 300 cycles	> 80 DF	Yes
Scaling resistance	ASTM C672	28 days — 0 to 5 visual rating at 50 to 300 cycles	< 2	Yes
	ASTM C672 modified by ICRI 320.2R		Committee 546 does not have a recommended value for this test.	Yes
3.6.2—Permeability				
90-day ponding	AASHTO T259	0.42 percent at 0.5 in. (13 mm) 0.15 percent at 1.0 in. (25 mm)	Committee 546 does not have a recommended value for this test.	Yes
	ASTM C1543			No
Rapid chloride permeability	AASHTO T277	28 days – 4000 to 5000 C	4000 C‡	Yes
	ASTM C1202			Yes
Absorption after immersion	ASTM C642	4 to 6 percent	< 6 percent	Yes
Volume of permeable pore space	ASTM C642	5 to 12 percent	< 12 percent	Committee 546 does not have a recommendation for this test.
3.6.3—Alkali-aggregate reaction	ASTM C227	Refer to 3.6.3	< 0.1 percent	Yes
	ASTM C1260	Refer to 3.6.3	< 0.1 percent	Yes
	ASTM C1293	Refer to 3.6.3	< 0.1 percent	Yes
	ASTM C289	Refer to 3.6.3	Refer to 3.6.3	No; may not reliably predict aggregate reactivity in concrete.
	ASTM C295	Refer 3.6.3	Refer to 3.6.3	Yes
3.6.5—Abrasion resistance	ASTM C779 Procedure A	0.004 to 0.1 in. (0.1 to 2.5 mm) at 30 min. 0.008 to 0.2 in. (0.2 to 5.1 mm) at 60 min.	Refer to 3.6.5	Refer to 3.6.5
3.6.6—Sulfate resistance	ASTM C1012	0 to 0.2 percent	< 0.1 percent	Yes

*The recommended values are minimum values recommended by Committee 546 for typical repair materials used in successful repairs and are not necessarily applicable for all conditions.

†Material properties based on a specific modification to a test method discussed in this chapter.

‡Committee 546 recommends that test results be correlated with test results from AASHTO T259 or ASTM C1543.



Documents

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Table 3.8b—Summary of changes in material properties of basic portland-cement replacement and overlay concrete when adding various admixtures, polymer modifiers, and pozzolans; using other cements; or removing coarse aggregates

Modification to basic portland-cement concrete	Effects on properties
Various hydraulic cements (slag cement is discussed separately below)	Reduce heat of hydration; reduce set time; increase early strength; reduced drying shrinkage; expansive or shrinkage compensating; improve sulfate resistance.
<i>Chemical admixtures</i>	
Air-entraining	Improved workability; improved durability in freezing-and-thawing, deicing, sulfate, and alkali-reactive environments; reduced compressive strength.
Accelerating	Accelerated set and early-strength development; increased creep and drying shrinkage.
Water-reducing, mid-range water-reducing	Reduce water content at least 5 percent (5 to 10 percent for mid-range); reduced permeability; increased strength.
Water-reducing and set-retarding	Reduce water content at least 5 percent and delay set time; reduced permeability; increased strength.
High-range water-reducing	Reduce water content by at least 12 to 40 percent, increase slump, decrease placing time, and increase flowability; reduced permeability; increased strength.
Flowing concrete	Slump greater than 7-1/2 in. (190 mm) while maintaining cohesive nature (ASTM C1017); reduced permeability.
Self-consolidating concrete	Reduced construction time and labor; improved formed surface finish; increased strength, shrinkage, and creep; reduced permeability and improved durability.
Very high early strength	Reduced set/working time; Strategic Highway Research Program (SHRP) SHRP-C-363 defines very high-early strength material as 2000 psi (13.8 MPa) minimum compressive strength 6 hours after mixing; durability factor of 80 percent after 300 freezing-and-thawing cycles in accordance with ASTM C666, Procedure A.
Extended set control	Used to stop or severely retard cement hydration process.
Shrinkage-reducing	Increased set time; reduce drying shrinkage by 30 to 50 percent; reduced strength, thermal cracking, and slab curling; increased susceptibility to freezing-and-thawing deterioration.
Corrosion-inhibiting	Significantly reduce rate of steel corrosion and extend time of onset of corrosion; reduced or increased compressive strength; may distort results of ASTM C1202 and AASHTO T277.
Lithium	May affect set/working time; minimize deleterious expansion from alkali-silica reaction.
Permeability-reducing: For non-hydrostatic conditions.	Water-repellent surface, reduced water absorption; Can affect finishing properties, consistency, compressive strength, freeze and thawing resistance, and shrinkage.
Permeability-reducing: For hydrostatic conditions.	Reduced permeability, increased resistance to water penetration under pressure; Can affect finishing properties, consistency, compressive strength, freezing-and-thawing resistance, and shrinkage.
Bonding	Increased bond, tensile, and flexural strength; decreased compressive strength; latex may cause excessive entrained air; some polymers may decompose and soft in the presence of moisture.
Rheology- and viscosity-modifying; Anti-washout	Increased cohesiveness; reduced segregation and bleeding; used as pumping aid, for concrete to be pumped underwater; reduced loss of cementitious material due to washout.
<i>Supplementary cementitious materials</i>	
Fly ash	Reduced water demand; reduction in portland cement; increased air-entraining admixture demand; improved workability; slower rate of reaction; reduced permeability and alkali-aggregate reactivity; improved sulfate resistance; discoloration of concrete.
Silica fume	Increased water demand; increased air-entraining admixture demand; decreased workability; increased cohesiveness; reduced bleeding; increased plastic shrinkage cracking; darker color; increased compressive and bond strength; increased electrical resistivity; reduced permeability; increased resistance to alkali-silica reaction; sulfate and chemical attack; increased abrasion resistance.
Natural pozzolans	Improved workability and finishing; reduction in portland cement; increased cohesiveness; increased set time; reduced permeability; increased strength at later ages; improved resistance to alkali-silica reaction in minimum dosages; too little natural pozzolan may actually increase detrimental effects of alkali-silica reaction; improved resistance to sulfate attack.
Slag cement	Improved workability; increased set time; reduced early rate of heat generation; reduced strength at early ages; increased strength at later ages; lighter in color; reduced permeability and potential expansion due to alkali-silica reaction; improved sulfate resistance.
Polymer modifiers	Excessive amounts of entrained air unless antifoam agent is used; improved workability; increased set time; increased tendency for plastic-shrinkage cracking; lower compressive strength; increased bond and tensile strength; reduced permeability; improved resistance to freezing and thawing; may improve impact strength and abrasion resistance.
<i>Fiber reinforcement</i>	
Steel fibers	Reduced slump and workability; tendency for fibers to ball; increased number of smaller-width shrinkage cracks; improved post-cracking ductility; increased compressive, direct tension, shear and torsion, flexural, and flexural fatigue strength; improved toughness and resistance to flexural impact loading.
Synthetic fibers	Reduced workability; reduced average width of shrinkage cracks; improved post-cracking ductility; improved resistance to impact, fatigue strength, and flexural toughness.
Polymer concrete	Increased safety concerns during construction; typically less contractor familiarity with materials and installation procedures; reduced cure time; reduced weight; increased tensile, flexural, compressive, and bond strengths; reduced modulus of elasticity; increased coefficient of thermal expansion; reduced permeability; improved chemical resistance.
Portland-cement mortar	Increased drying shrinkage; may be modified by substituting various hydraulic cements or polymers; or including chemical admixtures, mineral admixtures, polymer modifiers, slag cement and fiber reinforcement, or both; effects on properties are similar to those discussed for concrete above.

The materials most commonly used for concrete replacements and overlays are:

- a) Portland or blended cement-based concrete;
- b) Portland or blended cement-based, silica-fume concrete (silica-fume concrete), sometimes with fiber reinforcement;
- c) Portland or blended cement-based, polymer-modified concrete;
- d) Magnesium-ammonium-phosphate-cement concrete (MAPCC);
- e) Polymer-based concrete (polymer concrete);
- f) Portland or blended cement-based mortar (cement mortar);
- g) Portland or blended cement-based, silica-fume mortar (silica-fume mortar), sometimes with fiber reinforcement;
- h) Portland or blended cement-based, polymer-modified mortar;
- i) Magnesium-ammonium-phosphate-cement mortar (MAPCM);
- j) Polymer-based mortar (polymer mortar).

Before any repairs are undertaken, it is essential to follow the assessment steps (1.2) to achieve satisfactory results.

4.1—Concrete

Concrete is composed of portland or blended cement, fine and coarse aggregates, and water. Admixtures are frequently used to entrain air, accelerate or retard hydration, improve workability, reduce mixture water requirements, increase strength, or alter other freshly mixed and hardened properties of concrete. Pozzolanic materials, such as some fly ashes, may be used in conjunction with portland cement for economy or to affect specific properties, such as reduced early heat of hydration, increased later-age strength development, reduced permeability, or increased resistance to alkali-aggregate reaction and sulfate attack.

Concrete proportions should be carefully selected to provide the necessary characteristics for the specific application, such as workability, density, strength, durability, or resistance to deterioration under the expected exposure conditions (ACI 211.1). To minimize drying shrinkage, repair concrete should have the lowest possible water content, the smallest paste volume, and the largest coarse aggregate size and highest coarse aggregate content that are feasible. Shrinkage-reducing admixtures or shrinkage-compensating cements may also be considered when proportioning a concrete mixture. According to ACI 201.2R, frost-resistant, air-entrained, normalweight concrete should have a maximum w/cm of 0.45 for thin sections and 0.50 for all other structures. Mixing, transporting, placing, and curing should follow the guidance of ACI 213R, ACI 304R, ACI 304.1R, ACI 304.2R, ACI 304.6R, and ACI 308R.

Many properties of rebuild and overlay concrete such as modulus of elasticity and coefficient of thermal expansion are typically of similar magnitude to those of the substrate concrete. Entrained air can be incorporated into the repair concrete to provide resistance to freezing and thawing. The quality of concrete, particularly its permeability and enhanced corrosion protection of embedded reinforcement, is frequently improved by reducing the w/cm to 0.40 or less in



Fig. 4.1—Ready mixed concrete being placed. (Courtesy of Wiss, Janney, Elstner Associates, Inc.)

conjunction with using a high-range water-reducing admixture. This type of concrete is called low- w/cm concrete.

A concrete or mortar repair will usually contract due to plastic and drying shrinkage, cooling, and autogenous volume changes. If this contraction is restrained through bond to a stable substrate, tensile strains develop in the repair material. When these strains exceed the tensile strain capacity of the hardened concrete, cracks develop. This cracking may require further repair or future maintenance because of the possibility of ingress of deleterious materials, loss of bond to substrate, edge spalling of the crack under traffic conditions, or aesthetic objections. Drying shrinkage can be minimized with appropriate mixture proportioning, such as maximizing the size of coarse aggregates compatible with the depth of the application. Plastic shrinkage cracking can be minimized with timely application of curing materials. Some admixtures, such as accelerating admixtures, may increase the repair material shrinkage and the resulting cracking. While properly proportioned concrete has good durability, other repair materials, such as polymer concrete and magnesium-ammonium-phosphate concrete, are more resistant to certain types of chemical attack. Ready mixed concrete being placed is shown in Fig. 4.1.

4.2—Silica-fume concrete

Silica fume, a by-product of the ferrosilicon industry, is a highly pozzolonic material used to enhance mechanical and durability properties of concrete. Silica-fume concrete is conventional concrete with silica fume and a high-range water-reducing admixture added. Typical silica fume dosage ranges from 5 to 10 percent replacement of cement by mass.

Silica-fume concrete has properties similar to, but generally stronger than, conventional concrete, including higher bond strength. It also has lower permeability and provides increased protection against corrosion of embedded reinforcement. It is more resistant to attack by some chemicals and to the abrasive action of waterborne solids in hydraulic structures with high flow rates in confined areas (ACI 234R).

Silica-fume concrete is more cohesive and has less bleed water than conventional concrete. Consequently, it is more difficult to finish and, without proper protection during placement and timely curing, more susceptible to plastic

shrinkage cracking than concrete with a moderate w/cm . The potential for cracking of restrained concrete repairs, with and without silica fume, should be recognized. Any variations in concrete materials, mixture proportions, and construction practices that minimize shrinkage or reduce temperature differentials should be considered. Cracking due to restrained shrinkage or temperature movements may necessitate further repair. Silica-fume concrete has a gray to black color and may not blend in well with the appearance of the adjacent existing concrete (ACI 234R).

4.3—Polymer-modified concrete

Polymer-modified concrete is a portland or blended cement-based concrete with polymer modifiers added. Typical polymer modifiers include styrene butadiene, acrylic, vinyl acetate-ethylene, styrene acrylic, and epoxy. Generally, the polymer modifier is added at the rate of 10 to 20 percent by mass of cement. The w/cm for concrete with a latex modifier is normally 0.30 to 0.40, including the water in the latex, and 0.25 to 0.35 for concrete with an epoxy modifier.

Compared with conventional concrete, polymer-modified concrete has increased flexural and bond strength, reduced permeability, and provides increased protection against corrosion of embedded reinforcement (Mindess and Young 1981). Polymer-modified concrete is more difficult to place and finish than conventional concrete, and has a relatively short working time—approximately 15 to 30 minutes—before the dispersed polymer begins to coalesce. Although it can still be worked after the polymer has begun to coalesce, the beneficial properties of the polymer modification will be significantly reduced. The mixture should never be retempered. The manufacturers' instructions may not recognize the constructibility limitations as described above (ACI 548.1R).

Because of the short working time, polymer-modified materials should be batched and mixed on site. When a concrete mixer is used on small repairs, batch sizes should be restricted to quantities that can be placed and finished within the working time. A longer mixing time may result in a marked increase in total air content, which can result in a significant reduction in the compressive strength. Because the manufacturer may or may not add a defoaming agent to the polymer, they should be consulted for the recommended mixing procedures and times. Mobile mixers are frequently used where large quantities of polymer-modified concrete are required. It is recommended that air content tests be performed.

Polymer-modified concrete is normally moist cured for 1 to 2 days, followed by air-drying. While the drying shrinkage of concrete is not increased by the addition of polymers, polymer-modified concrete is more sensitive to plastic shrinkage if curing material is not applied as soon as practical. Concrete modified with styrene butadiene or some epoxies will exhibit color change when exposed to ultraviolet light (ACI 548.1R and 548.3R).

4.4—Magnesium-ammonium-phosphate-cement concrete (MAPCC)

MAPCC is concrete in which magnesium-ammonium-phosphate is substituted for the portland or blended cement. It has excellent bond strength, low drying shrinkage, rapid strength gain, and low permeability (Popovics and Rajendran 1987; Popovics et al. 1987).

Because the binder is not based on portland cement, surface preparation, application, and curing requirements are different. MAPCC should not be applied over a carbonated substrate; that is, one with a pH that is no lower than 10 because low bond strength may result. The pH of the substrate should be tested before application. MAPCC hardens rapidly and generates ammonia and large quantities of heat during curing. Aggregates are used for large or deep placements to control the exothermic reaction; however, the aggregates should not contain carbonate minerals to avoid an undesirable chemical reaction with the binder. Placement procedures should avoid trapping ammonia gas within the MAPCC. The only necessary curing is a light application of curing compound in extreme drying conditions, such as direct sunlight, strong winds, high temperatures, or low humidity. Moist curing is not recommended.

MAPCC is used for specialized applications. The rapid strength gain and volume stability make this repair material a good option for fast-turnaround applications and for long, slender repairs. Hot and cold weather formulations are available. MAPCCs are excellent for cold weather applications. As long as the mixing water is above freezing, the exotherm as the product sets, as well as the ammonia in the product, prevent product freezing. Lower temperatures will slow strength development. Typically, the drying shrinkage of MAPCC is less than that of other rapid-setting repair materials. Contamination of MAPCC with other hydraulic cement or contaminated mixing water may result in low strength, rapid stiffening, and poor performance. The specifier and user should consult with the material manufacturer for specific material properties to determine the applicability and use of the material.

4.5—Polymer concrete

Polymer concrete is one in which an organic polymer serves as the binder. Polymers typically used include epoxy, polyester, furan, vinyl ester, or methyl methacrylate. Portland cement, however, is sometimes used as a filler (ACI 548.1R).

Polymer concrete may have low shrinkage as it cures, good bond strength to the substrate concrete, high tensile and flexural strength, low permeability, increased protection against corrosion of embedded reinforcing steel, and good resistance to chemical attack (ACI 548.1R). It also sets quickly, reducing the down time of the repair area. Because of the rapid setting time, however, it should be batched on site (Concrete Society 1976).

The cure time of polymer concrete is directly related to the polymer type and other agents used, concrete substrate temperature, air temperature, and temperature attained by the mixed polymer. While polymer concrete cures much more quickly than portland cement-based concrete,

low temperatures or an inappropriate polymer selection may significantly increase the set time. Conversely, high temperatures may reduce the working time below acceptable levels.

Polymer concrete has a significantly higher coefficient of thermal expansion than the substrate concrete, and in situations where the temperature of the service environment is not controlled, the aggregates in the polymer concrete mixture should be carefully graded and proportioned to minimize the distance between the aggregate particles, decreasing the coefficient of thermal expansion (Sprinkel 1983); preplacing the aggregates results in tightly packed aggregate. It is desirable to place polymer concrete near the midrange of service temperatures to reduce cracking and internal stress caused by long-term thermal stress cycles. In many instances, however, some cracking should be anticipated, and further repair may be needed.

Polymer concrete can also have a modulus of elasticity significantly different from the substrate concrete. Many polymer concretes have a lower modulus of elasticity that can be beneficial in reducing stresses that develop from other differences in the material properties, particularly the coefficients of thermal expansion. This, however, may also make this material inappropriate for structural repairs.

Most polymers used in polymer concrete soften when heated. Their mechanical properties change significantly beyond their heat deflection temperatures (HDT). Although the HDT varies for each formulation, for those systems used in concrete construction, it generally ranges from 60 to 160°F (16 to 71°C). The polymer formulation may need to be adjusted to achieve proper curing at higher temperatures. Some polymers may soften when exposed to high temperatures in fires, or solvents in surface coatings and sealers.

General construction workers usually are not experienced in working with polymer concrete and will require special training. Moisture in the aggregates or on the concrete surface adversely affects polymer concrete. Special clean-up solvents are necessary; these solvents may be hazardous materials that require special disposal procedures. Special safety equipment and precautions may be necessary due to fumes and flammability of some materials. Polymer concrete usually does not blend in with the appearance of the adjacent existing concrete (ACI 548.1R).

Because the properties of polymer concrete are significantly different from those of the substrate concrete, and because polymer concrete has higher material and installation costs, it is commonly used only for repairs with unusual requirements or in demanding environments where the enhanced properties of polymer concrete are desired.

4.6—Mortars

4.6.1 Cement mortar—Cement mortar generally consists of portland cement, fine aggregate, and water, and may contain other ingredients such as admixtures. Cement mortar is frequently mixed in small batches at the site; therefore, it may be difficult to maintain uniform characteristics. Because of the absence of coarse aggregate, higher water volume, higher cement content, and the higher paste-aggregate ratio, cement mortar shrinks more than concrete, often resulting

in more cracking, and further repair may be needed. Water-reducing admixtures, expansive agents, and other modifiers are sometimes used to reduce shrinkage. The coefficient of thermal expansion is dependent on aggregate content, type, and size, and on the cement content; therefore, some differences in thermal properties between mortars and concrete are likely (Neville 1996).

4.6.2 Prepackaged repair mortars—Prepackaged repair mortars are also commercially available. Hydraulic cement and other ingredients, including fine aggregate, dry admixtures, and, frequently, proprietary ingredients, are blended in a production plant and then packaged, commonly into several sizes of containers. Prepackaged materials may offer more consistent and predictable work-site performance. Additional potential advantages include, but are not limited to:

a) **Better mixture and quality control at the work site**—Commonly, only water is added at the work site.

b) **Quality control**—During production, most manufacturers inspect and test their materials to confirm that the material meets their product specifications before the material leaves the plant. Some manufacturers are certified by the International Organizations for Standards (ISO), which means that their production processes and procedures are documented and consistent with this documentation. Note, however, that ISO certification indicates a product is produced consistently, but does not imply that it is a good product.

c) **Performance-based materials**—Mixture proportions, often including admixtures or proprietary ingredients, are designed to meet specific needs, such as approval for contact with potable water (NSF certification), resistance to freezing and thawing, fast set, abrasion resistance, and low shrinkage.

d) **Data readily available**—Performance test results are usually available from the manufacturers. Product data sheet information provides benchmarks for the applicator and specifier to compare with on-site testing; however, differences in test methods may create difficulties comparing properties between materials. Modified test procedures can also produce misleading performance expectations.

e) **Those mixtures that can be extended with coarse aggregate to create concrete for repairing deeper sections**—The extended mixtures will have properties more similar to concrete rather than mortar.

These prepackaged materials have a limited shelf life, and generally should not be used after the expiration of this limit.

4.6.3 Silica-fume mortar—Silica-fume mortar is cement mortar with silica fume added. Silica fume increases the bond strength and cohesiveness of mortar. Silica-fume mortar also has increased compressive and tensile strength, reduced permeability, increased electrical resistivity characteristics, and is also more resistant to chemical attack. Again, the absence of coarse aggregate causes increased shrinkage and greater tendency to crack.

The potential advantages outlined in 4.6.2 resulting from the use of prepackaged materials apply, plus the benefits associated with the incorporation of silica fume into a repair mortar. Site-added silica fume may be difficult to uniformly disperse.