Technical Report REMR-CS-47
April 1995

US Army Corps
of Engineers
Waterways Experiment
Station

*Repair, Evaluation, Maintenance, and Rehabilitation Research Program*

# Performance Criteria for Concrete Repair Materials, Phase I

*by Peter H. Emmons, Alexander M. Vaysburd,*
*Structural Preservation Systems, Inc.*







Prepared for Headquarters, U.S. Army Corps of Engineers

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| | Problem Area | | Problem Area |
|---|---|---|---|
| CS | Concrete and Steel Structures | EM | Electrical and Mechanical |
| GT | Geotechnical | EI | Environmental Impacts |
| HY | Hydraulics | OM | Operations Management |
| CO | Coastal | | |

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Final report

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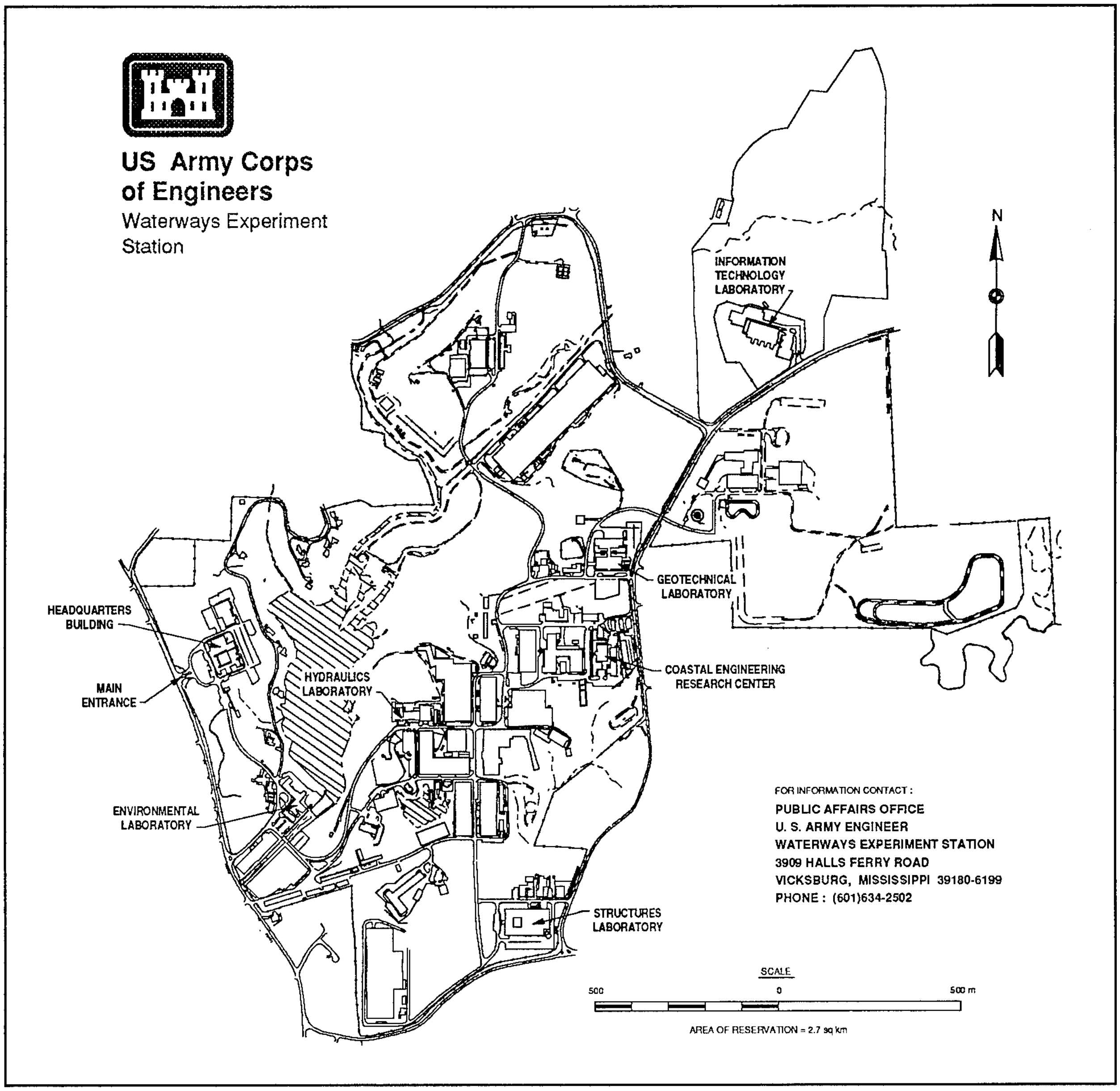


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# Preface

The study reported herein was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), under Civil Works Research Work Unit 32637, "Evaluation of Existing Repair Materials and Methods," for which Mr. James E. McDonald, U.S. Army Engineer Waterways Experiment Station (WES), Structures Laboratory (SL), is Principal Investigator. This work unit is part of the Concrete and Steel Structures Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program sponsored by HQUSACE, for which Mr. McDonald is the Problem Area Leader.

The REMR Technical Monitor is Dr. Tony C. Liu (CECW-EG), HQUSACE. Mr. William N. Rushing (CERD-C) is the REMR Coordinator at the Directorate of Research and Development, HQUSACE. Mr. James E. Crews (CECW-EG) and Dr. Liu serve as the REMR Overview Committee. Mr. William F. McCleese, WES, SL, is the REMR Program Manager.

The study was performed by Structural Preservation Systems, Inc., Baltimore, MD, under contract to WES. The work was conducted under the general supervision at WES of Mr. Bryant Mather, Director, SL, and Dr. Liu, Acting Chief, Concrete Technology Division, and under the direct supervision of Mr. McDonald. This report was prepared by Messrs. P. H. Emmons and A. M. Vaysburd, Structural Preservation Systems, Inc.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

# Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

| Multiply | By | To Obtain |
|---|---|---|
| Fahrenheit degrees | 5/9 | Celsius degrees or Kelvins[1] |
| cubic yards | 0.7645549 | cubic metres |
| gallons (U.S. liquid) | 3.785412 | litres |
| feet | 0.3048 | metres |
| inches | 25.4 | millimetres |
| kips (force) | 4.448222 | kilonewtons |
| pounds (force) | 4.448222 | newtons |
| pounds (force) per square inch | 0.006894757 | megapascals |
| pounds (mass) | 0.4535924 | kilograms |
| pounds (mass) per cubic yard | 0.5932764 | kilograms per cubic metre |
| pounds (mass) per square foot | 4.882428 | kilograms per square metre |
| square inches | 0.0006451 | square millimetres |

[1] To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use $K = (5/9)(F - 32) + 273.15$.

# 1 Introduction

## Project Description

Significant properties affecting the dimensional compatibility of a repair with an existing concrete substrate in a repair system were defined.

Preliminary performance criteria for concrete repair materials were developed for screening and selecting cement-based surface repair materials based upon their dimensional compatibility with existing concrete. The property of drying shrinkage was one of the most critical properties affecting performance. A classification was developed for cement-based repair materials into four categories based upon their drying shrinkage.

The preliminary criteria were developed based on shrinkage classification of repair materials and information developed as a result of a comprehensive search of available information. Preliminary requirements for acceptable performance of repair materials in repaired structures in terms of the results of this study were established. Difficulties with specifying materials based on their shrinkage have resulted not only from the lack of acceptable performance criteria but also from to the lack of a reliable industry-wide shrinkage test standard. The criteria are based on American Society for Testing and Materials (ASTM) C 157 (ASTM 1994e) test results because this test method was, at this time, the most used shrinkage test method for which performance data exists.

The criteria are preliminary because they are based on very limited laboratory unrestrained shrinkage test results. Laboratory and field performance data need to be obtained for other test methods with potentially better precision and correlation with in situ performance. The experimental program is developed to verify the preliminary performance criteria and to select a reliable shrinkage test method.

Most likely, the criteria must be modified as additional laboratory and field performance results are obtained.

## Background

Concrete repair and restoration is considered the growth sector of the construction industry of the 1990's. It was only recently estimated that in the United States nearly $50 billion will be necessary to repair or restore currently deficient bridges. Repair of hydraulic structures, (dams, spillways, lock chambers), buildings, parking structures, and other concrete structures will substantially increase this figure.

The Corps of Engineers operates and maintains approximately 600 dams and 280 lock chambers. Of these, more than 40 percent are over 30 years old and 29 percent were constructed before 1940. With relatively little new construction anticipated, many of these structures will be kept in service well beyond their original design lives by extensive maintenance, repair, and rehabilitation. Considerable savings in the cost for the repair and rehabilitation of deteriorated concrete structures would be realized if performance criteria for the selection of repair materials compatible with existing concrete substrate could be developed. To this end, Structural Preservation Systems, Inc. (SPS), has carried out a program entitled "Performance Criteria for Concrete Repair Materials."

Durability of concrete repairs, to a large degree, depends on the correct choice and use of repair materials. Restrained contraction of repair materials, the restraint being provided through bond to the existing concrete substrate, is a major factor which significantly increases the complexity of repair projects as compared to new construction.

Selecting repair materials requires an understanding of material behavior in anticipated service and exposure conditions. One of the greatest challenges facing successful performance of repair materials is their relative dimensional behavior to the substrate. Relative dimensional changes cause internal stresses within the repair material and the substrate. High internal stresses may result in tension cracks, loss of load-carrying capacity, delamination, and deterioration. Particular attention is required to minimize these stresses and select materials that properly address relative dimensional behavior. Finding materials that behave the same as the substrate when subjected to loads, temperature, and moisture changes is unlikely. The requirement for durable repairs is that selected repair materials must have properties dimensionally compatible with the substrate.

However, it should be indicated that having a durable repair material does not guarantee that the repair system (existing structure-interface-repair) will not fail. The repair material must protect the existing concrete from the agents that caused the deterioration. For instance, if the in situ concrete is nonair-entrained and the deterioration is caused by freezing and thawing, the repair must protect the in situ concrete from becoming critically saturated or from freezing. The chances of protecting against saturation are very unlikely. Therefore, the repair must be capable of protecting the in situ concrete from freezing. If the existing concrete continues to deteriorate from the original cause, bond will be lost between the repair and in situ concrete and the repair will fail.

At present, there is little information on the durability of repair systems. The choice of material is made largely on information from the manufacturer or on the guesswork and experience of what has in the past been adequate. This "process" impedes improvement.

While the economics and difficulties of carrying out repairs provide a strong argument for researching the durability of repair systems, there are some difficulties. These stem from the following factors:

*a.* Each of the broad categories of repair materials has a wide variation of properties within it so that there are no representative materials.

*b.* Realistic durability testing requires representative repairs to be exposed to a real world environment for realistic durations.

*c.* Repair materials are continuously under development. By the time durability studies have been completed, materials have already been changed.

However, these arguments are common to all research. Research on a few reasonably representative repair materials will provide valuable basic information on the parameters controlling durability. It will also provide benchmarks against which the properties of more recently developed materials can be judged. Research on the durability of repair systems will improve the selection of materials and will result in repairs being carried out more efficiently. It will also encourage the improvement of repair materials and systems as more is learned about the mechanisms involved and as the important parameters affecting durability are identified.

At present, there is very limited information on the durability of repairs. Consequently, there is no yardstick by which to judge the choice of repair methods or materials. Our study of manufacturers' data sheets indicates that some known characteristics of selected products have been recorded. However, only brief property details are listed on data sheets otherwise packed with descriptions of methods for preparation and placement. More detailed information on ingredients and relevant technical data regarding tensile strengths, shrinkage, adhesion strength to both concrete and steel, strain capacity, and creep are invariably unavailable, unresearched, or secret. However, compressive strength is quoted.

With the variety of ways now possible to achieve a given level of performance, including durability, there is considerable pressure to develop and use performance criteria. Unfortunately, development of such criteria has not kept pace with the development of materials, primarily because of the lack of appropriate scientific and field data needed for its development.

Considering that at the present time the designer and contractor have a greater choice of materials and methods, a much more sophisticated approach to addressing, controlling, and specifying durability is needed. Specifically required is a more direct means of linking the properties of the cement-based materials and the production practices with the quality and performance of what is actually produced.

Development of an adherence to sound specifications is one of the avenues to improved durability. Introduction of performance criteria will require improved understanding of the relationships between the composition, microstructure, and physical performance of cement-based composites. Dimensional compatability between a repair and an existing structure is a milestone of such criteria. One of the most important properties affecting dimensional compatibility is drying shrinkage.

To specify the appropriate material, to verify manufacturers' claims, and to evaluate performance of products is almost impossible at this time due to the variety of methods which measure the shrinkage of materials. In addition to the absence of a reliable industry-wide testing method, manufacturers who are using the same standard method, for instance ASTM C 157, "Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete," are arbitrarily modifying the method. The arbitrary application of test methods has resulted in controversy and confusion in selecting and specifying materials. It also hinders the selection of the method most applicable for a given generic type of material. Variations in the test methods, including size of the specimen, restraint conditions, curing, time of initial readings, temperature and relative humidity limitations, and test duration further complicate the interpretation of comparative test results and properties indicated on manufacturers' data sheets. As a result of the arbitrary application of test methods, the ability to predict future performance of many materials is weakened.

It has become clear that test results must be carefully scrutinized to ensure their validity and to avoid misrepresentation. Because there are still no internationally accepted standards by which to judge the quality and performance of repair materials, suppliers often question the necessity and validity of individual tests. The establishment of formalized and authorative procedures and criteria in national and international standards and codes of practices is still the ultimate ideal. A system must be developed to address specific needs in specific circumstances, particularly with regard to the types of repairs carried out and the practical and climatic conditions under which they are executed and required to perform.

## Purpose, Scope, and Approach

The general objective of this study was to develop performance criteria for selecting cement-based repair materials. The purpose of the current study

(Phase I) was to develop preliminary performance criteria for dimensionally compatible repair materials and to select a reliable shrinkage test method for use in laboratory/field investigations. Finally, the scope of Phase I also included the development of an experimental program for laboratory and field tests to verify the preliminary performance criteria in Phase II of the project (Figure 1).

Preliminary performance criteria were developed based on a state-of-the-art review of factors affecting durability of concrete repairs and establishing and characterizing properties affecting dimensional compatibility of repair materials with existing concrete.

The critical review of manufacturers' data sheets for cementitious and modified cementitious repair materials commonly used in concrete surface repairs indicated an absence of complete information on some of the critical properties of repair materials, such as shrinkage and creep. In many cases when shrinkage was indicated, various standard, nonstandard, and modified standard methods were used for shrinkage testing. These results are not comparable. Therefore, the tabulation of data for those properties was eliminated from further consideration because of the failure to get comparable information. Personal contacts and visits were also made with the leading material manufacturers such as Master Builders Technologies, Sika Corporation, Thoro System Products, Fosroc, Euclid, and other various groups active in repair materials research and evaluation.

Several field visits were made to survey the effect of various climatic conditions on repairs performed using the same materials. A classification of repair materials based on their shrinkage is proposed.

Available shrinkage test methods from the United States and abroad were evaluated, and the advantages and the limitations of each method were identified. Three methods currently being used and one developed by SPS are selected for evaluation in Phase II of the project.

A recommended experimental program for evaluating the field performance of the four categories of repair materials (very low, low, moderate, and high shrinkage) and verifying the preliminary performance criteria is presented for implementation in Phase II of this research program.

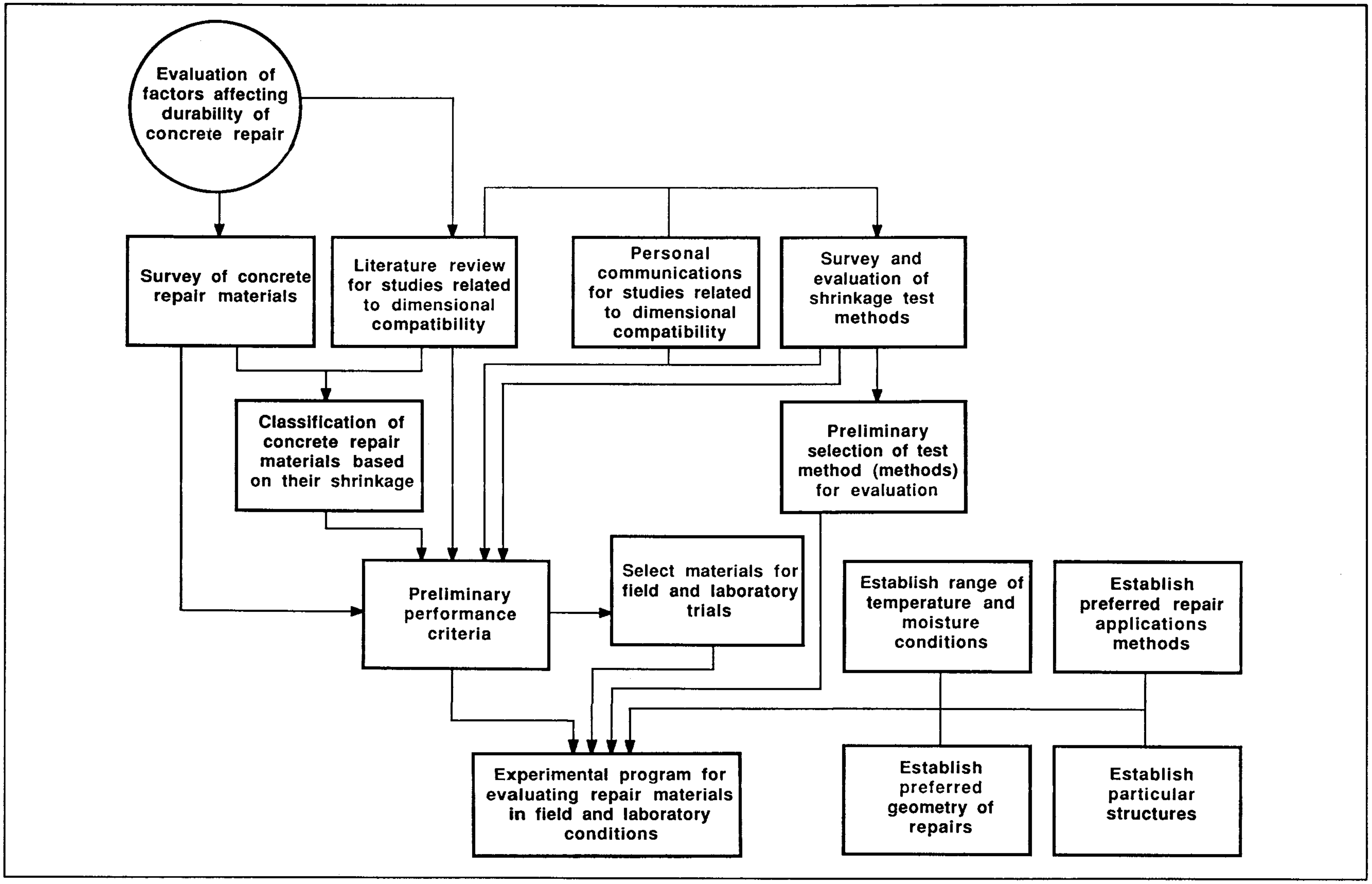



Figure 1. Flow chart of Phase I - Preliminary performance criteria and experimental program

# 2 Factors Affecting Durability of Concrete Repair

## The System Concept

The object of a repair is to produce a durable repaired structure with a limited and predictable degree of change without deterioration or distress throughout its intended life and purpose. Components of a durable repaired system are shown in Figure 2.

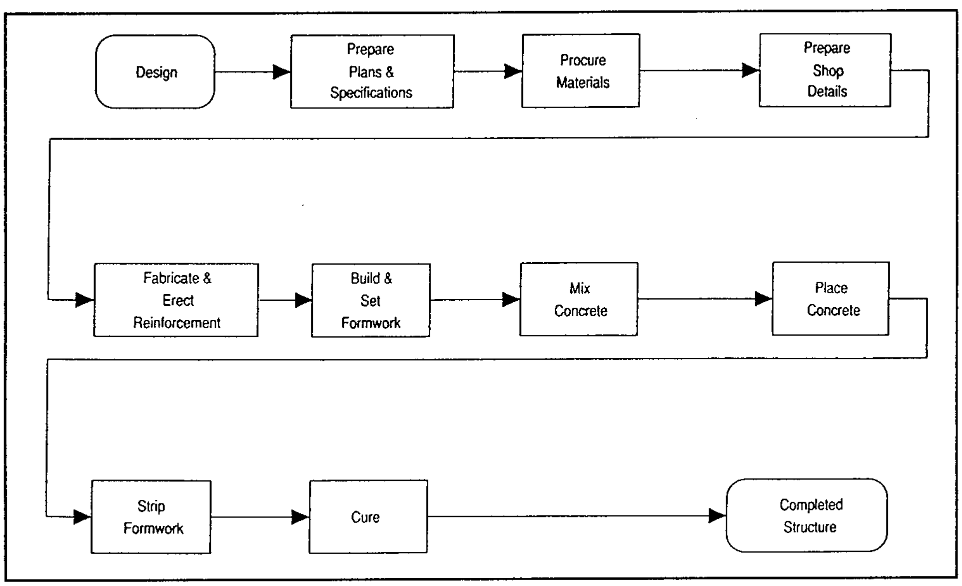



Figure 2. Components of durable repaired system

A study considering the repaired structure as a three-phase composite system (Figure 3) consisting of the existing concrete substrate, the repair material, and the transition zone between the two is proposed (Emmons and Vaysburd 1993).

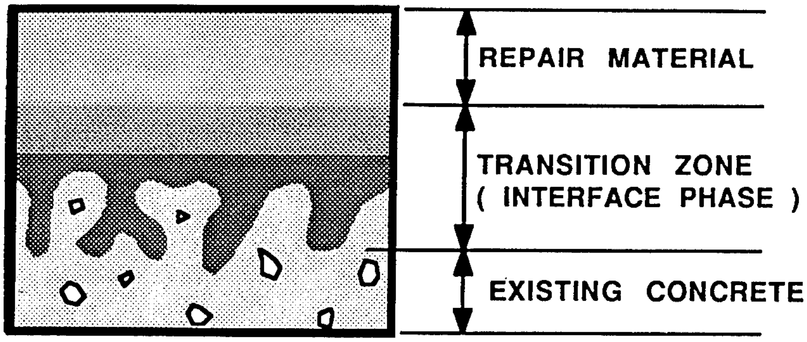



Figure 3. Idealized model of a surface repair system

The system concept can be derived from Aristotle's (350 B.C.) dictum: "The whole is more than just the sum of components." His definition could be treated as the "key" to the contemporary system theory (Kuhn 1962).

Selection of a repair material is one of the many interrelated steps in achieving a durable repaired structure. Equally important are the design and specifications, the construction practices, and the inspection.

Durability problems manifest themselves as spalling, cracking, delamination, scaling, and loss of strength - signs of distress that can and often do have multiple causes. Many of the factors that can affect durability are interrelated, making it difficult or impossible to identify any single underlying problem. Generally, a combination of factors is responsible. A combination of factors, therefore, must be considered in designing the repair. Critical factors affecting the durability of concrete repair system are presented in Figure 4.

Long service life of concrete repairs to a large degree depends on correct choice and use of repair materials. Restrained contraction of repair materials, the restraint being provided through bond to the existing concrete substrate, is a major factor which significantly increases the complexity of repair projects as compared to new construction. In recent years, a large number of different proprietary brands of repair materials have been used and new materials are being introduced continually. They represent the range of generically different repair materials and systems available and include cementitious, polymer-based, and polymer-modified cementitious materials. The precise formulation of the materials varies from one supplier to another, and even within one family group properties vary significantly. Table 1 illustrates typical differences in some of the important short-term properties of repair materials (Hanson and Yong 1991).

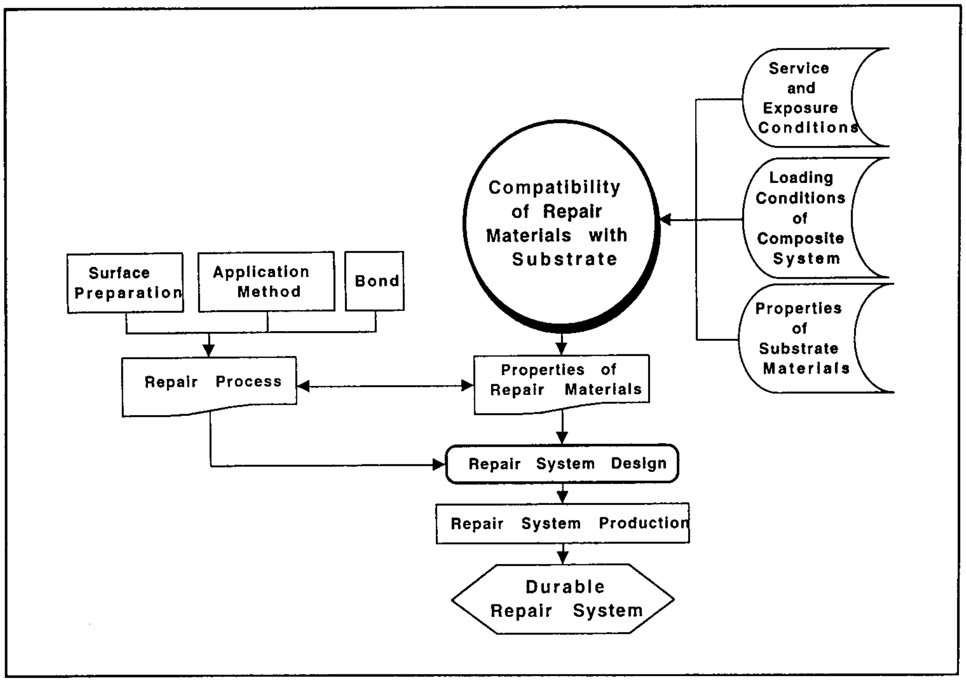



Figure 4. Factors affecting durability of concrete repair system

**Table 1**
**Typical Short-Term Properties of Repair Materials (from Hanson and Yong 1991)**

| Property | Resin Mortar | Polymer Modified Cementitious Mortar | Plain Cementitious Mortar |
|---|---|---|---|
| Compressive strength (psi/MPa) | 7250-14500<br>50-100 | 4350-8700<br>30-60 | 2900-7250<br>20-50 |
| Tensile strength (psi/MPa) | 1450-2175<br>10-15 | 72 -1450<br>5-10 | 290-725<br>2-5 |
| Modulus of elasticity in compression (psi/MPa) | $(1.45\text{-}2.9) \times 10^6$<br>$(10\text{-}20) \times 10^3$ | $(2.18\text{-}3.63) \times 10^6$<br>$(15\text{-}25) \times 10^3$ | $(2.9\text{-}4.35) \times 10^6$<br>$(20\text{-}30) \times 10^3$ |
| Coefficient of thermal expansion (millionths/°F millionths/°C) | $(14\text{-}17) \times 10^{-6}$<br>$25\text{-}30 \times 10^{-6}$ | $(6\text{-}11) \times 10^{-6}$<br>$10\text{-}20 \times 10^{-6}$ | $5.5 \times 10^{-6}$<br>$10 \times 10^{-6}$ |
| Water absorption (% by mass) | 1-2 | 0.1-0.5 | 5-15 |
| Maximum service temperature (°F/°C) | 104-177<br>40-80 | 215-571<br>100-300 | >571<br>>300 |

Some studies related to the selection of repair materials are advocating as a fundamental principle: "Repair like with like" (Building Research Establishment 1982, Department of Transportation, U.K. 1986, The Concrete Society 1984, Pullar-Strecker 1987). Wood, King, and Leek (1990) stated that "where the repair materials have ingredients compatible with the concrete in terms of cementitious mix content, aggregate-cement ratio and water-cement ratio (w/c), they achieve a match of "like with like; they can be approved without elaborate testing subject to site repair procedure trials." They also conclude that all repair materials which depart from similarity of mixture composition with concrete must be tested fully for dimensional properties so they can be formulated to match.

Another point of view is expressed by Plum (1990a), who calls the concept, "repair like with like," illogical. Plum states that "temptation to see parity of properties of the repair material and base concrete is strong, but attempts to avoid mismatch founder on the definition of compatibility." Further, Plum suggests that inspections of overlays, toppings, and patch repairs that have failed indicate that perfect parity of material properties will not necessarily prevent failure. This suggestion is confirmed by thermal compatibility tests in the United States (Mirza 1989), where cement mortar was an unsatisfactory material. The real requirement is that the repair material will have properties and dimensions which will make it compatible with the substrate for the application in hand. Plum considers the following material properties important for compatibility:

*a.* Elastic modulus

*b.* Creep and shrinkage

*c.* Bond strength

*d.* Temperature and humidity effects

Table 2 presents the point of view of Master Builders Technologies (1993) on required engineering properties of materials for different repairs and testing methods.

The present study and previous experience allows us to conclude that differences in properties will always exist between the repair material and the substrate concrete, regardless of the material. Even by using concrete as a repair material, it is impossible to match all properties of the substrate material, because at the time of the repair a large percentage of the ultimate shrinkage has already taken place.

Surface repair material needs are illustrated in Figure 5. Selecting repair materials requires an understanding of material behavior in the cured and uncured states in anticipated service and exposure conditions. One of the greatest challenges facing successful performance of repair materials is their relative dimensional behavior to the substrate. Relative dimensional change

## Table 2
## Required Engineering Properties of Concrete Repair Materials (Master Builders Technologies 1993)

| Required Engineering Properties | | | Repair | | | Floor |
|---|---|---|---|---|---|---|
| **Basic** | | | Horiz. | Vert. | Over-head | Overlay |
| Compressive Strength | ASTM C 109 | 2" cube | X | X | | X |
| Strain-Strain Compressive Strength<br>Modulus of Elasticity<br>Maximum Strain<br>Toughness | ASTM C 39 & C 469 Modified | 3" × 6" cylinder | X | X | X | X |
| Split Tensile Strength | ASTM C 496 | 3" × 6" cylinder | X | X | X | X |
| Punching Shear Strength | In-House | 3" × 4" × 11" beam | | X | | |
| Flexural Strength | ASTM C 78 | 3" × 4" × 16" beam | X | X | X | X |
| **Bond to Concrete** | | | | | | |
| Direct Tensile Bond | In-House | 3" × 6" cylinder | X | | X | X |
| Direct Shear Bond | Mich. DOT | 4" cube | | X | | |
| Flexural Bond | ASTM C 78 Modified | 3" × 4" × 16" beam | X | X | X | X |
| **Volume Change** | | | | | | |
| Length Change | ASTM C 157 | W" × H" × 11" beam | X | X | X | X |
| Coeff of Thermal Exp | ASTM C 531 | W" × H" × 11" beam | X | X | X | X |
| Creep | ASTM C 512 | 3" × 6" cylinder | | X | | |
| **Other** | | | | | | |
| Abrasion Resistance | ASTM C 779 | 12" × 12" × 1" slabs | X | | | X |
| Resistance to Freezing and Thawing | ASTM C 666 | 3" × 4" × 16" beam | X | X | X | X |
| Electrical Resistivity | In-House | W" × H" × 11" beam | X | X | X | X |
| Rapid Chloride Permeability | ASTM C 1202 | 4"D × 2"H Disk | X | X | X | X |
| Chloride Ponding | AASHTO T-259 | 12" × 12" × 4" slab | X | | | X |
| Exotherm | In-House | 3" × 6" cylinder | X | X | X | X |

Note: ASTM Standards and American Association of State Highway and Transportation Officials (AASHTO) information is listed in the references section following the main text.

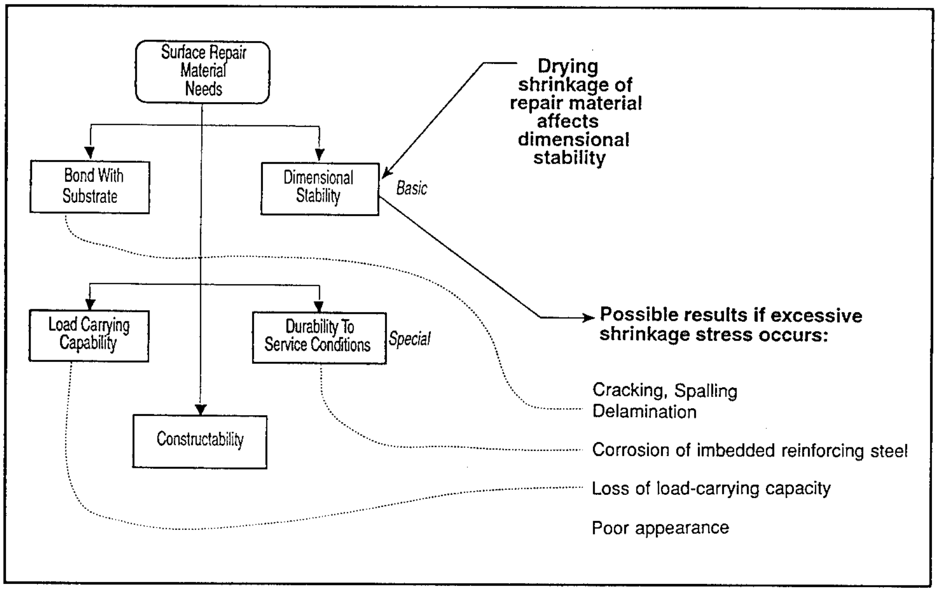



Figure 5. Surface repair material needs

causes internal stress within the repair material and in the substrate. High internal stresses may result in tension cracks, loss of load carrying capacity, delamination, and deterioration. Particular attention is required to minimize these stresses and to select materials that properly address relative dimensional behavior. Finding materials that have the same properties and behave the same during cure and hardened state as the substrate when subjected to loads, temperature, and moisture changes is unlikely. The requirement for durable repairs is that selected repair materials must have properties dimensionally compatible with the substrate to such a degree that the stresses induced at the interface and in the repair will not exceed tensile strength of the repair material. For the purpose of this study, we would like to define the term compatibility as it relates to the concrete repair.

Compatibility is the balance of physical, chemical, and electrochemical properties and dimensions between the repair phase and the existing substrate phase of a repair system. This balance ensures that the system withstands all anticipated stresses induced by volume changes, chemical, and electrochemical effects without distress and deterioration over a designed period of time.

As can be seen in the diagram presented in Figure 4, dimensional compatibility is one of the most critical components of concrete repair. Dimensional incompatibility adversely affects the durability of repairs or load-carrying capacity of structural repairs. In both cases, dimensional incompatibility may lead to failure. In cases of structural repairs, failure may be defined as an inability of the repair to carry the expected proportion of the load. Failure, by this definition, would not necessarily exhibit any outward signs of cracking or delamination. Consequences of incompatibility on the stress-carrying capacity of structural repairs is beyond the scope of the present study. Only problems of durability are under consideration in this study.

Critical material properties which influence dimensional compatibility include shrinkage, creep, modulus of elasticity, and thermal expansion (Figure 6). The research effort in this study is directed on establishing acceptable values of those properties for dimensionally compatible repair systems.

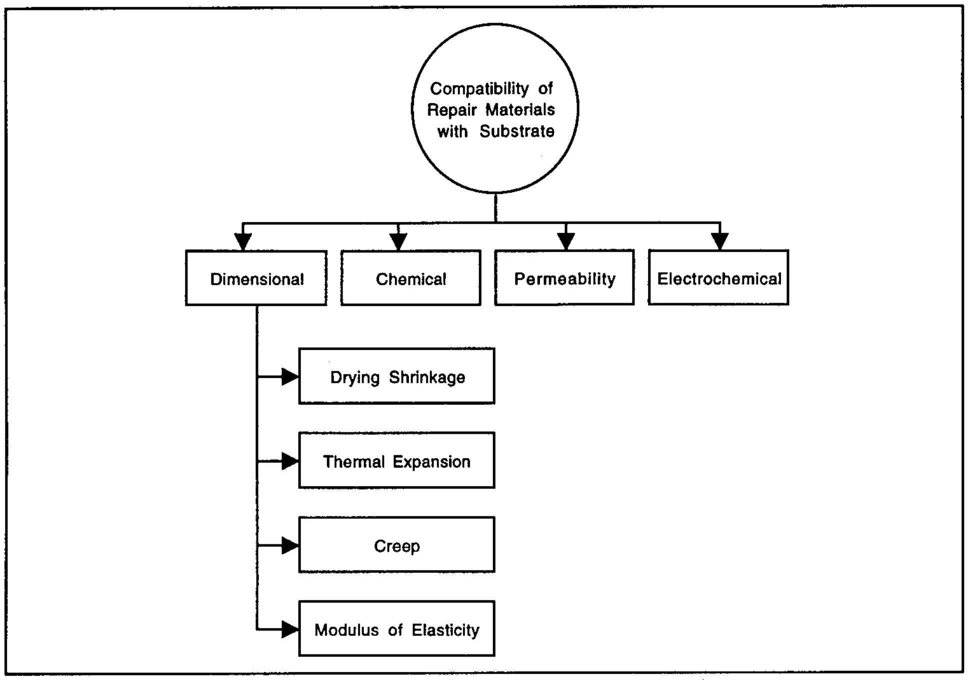



Figure 6. Factors affecting compatibility of repair materials

# 3 Properties Determining Dimensional Compatibility

## Volume Changes of Repair Materials

Deformations which may cause cracking in cement-based materials under service conditions arise from a number of different stimuli: applied stress, change of moisture content, and thermal changes. The response of the cement-based materials to these stimuli is complex, resulting in reversible, irreversible, and time-dependent deformations. This study concerns the dimensional compatibility of cement-based repair materials and, therefore, deals with four main types of deformations:

*a.* Shrinkage, which occurs upon loss of moisture from the material.

*b.* Creep, which is the time-dependent inelastic deformation that occurs with prolonged application of stress.

*c.* Strains due to temperature changes.

*d.* Instantaneous elastic deformations that occur when the external stress is first applied.

In each type, strains are of the same order of magnitude, so each type must be considered when specifying and selecting a repair material for a specific application.

Our literature studies and personal communications indicate that selection of repair materials based upon compatible thermal coefficients and moduli of elasticity is relatively easy to accomplish for cement-based materials, because they are more or less known properties. Shrinkage and creep are more critical time-dependent properties of cement-based materials and are often the least known.

The safety and durability of repaired structures cannot be realized without a comprehensive knowledge of the material's fundamental properties that determine its deformational characteristics. Among those are the creep properties under sustained loading and the shrinkage properties under drying conditions.

### Shrinkage and creep

Creep and shrinkage of cement-based materials are both long-term dimensional changes which are related to the movement of water within the hydration products and the microstructure of the paste and are modified by the composite nature of the cement-based materials. It is commonly stated in published literature and studies that shrinkage and creep are interrelated phenomena. They both originate from the same source - the hydrated cement paste.

Pertinent literature on creep and shrinkage mechanisms is reviewed. From this review, it can be demonstrated that creep and drying shrinkage cannot be accurately separated on concrete or other cement-based material specimens used for testing.

Perhaps the only distinction between creep and shrinkage resides in the mechanisms of basic creep (i.e., creep in material which has reached moisture equilibrium with the surroundings prior to creep loading). Numerous theories have been proposed to explain creep. Of these, the most prevalent creep hypotheses are: seepage theory, interlayer theory, thermally activated creep, and aging (Hanson and Yong 1991). No single hypothesis can fully explain the phenomenon of creep.

### Strain versus time

The strain-versus-time relationships for creep and shrinkage are very similar. The experimental parameters affect creep in a way similar to shrinkage. The magnitudes of the strains are similar and are of such magnitude that they affect durability and structural performances. However, it is a clear and justifiable tendency to deal with each phenomenon individually, since shrinkage is directly associated with moisture loss to the surroundings, and creep is not.

Shrinkage is defined as the time-dependent strain due to moisture loss at a constant temperature and in the absence of an external load.

Creep is defined as the total strain in a loaded specimen minus the initial elastic strain and the shrinkage in an unloaded companion specimen subjected to a similar environment:

$$Ec = Et - Ei - Es$$

where

$E_c$ = creep strain
$E_t$ = total time-dependent deformation
$E_i$ = initial elastic strain at time of application of load
$E_s$ = shrinkage strain

Figure 7 (Neville, Dilger, and Brooks 1983) shows schematically the terms and definitions involved in shrinkage and creep.

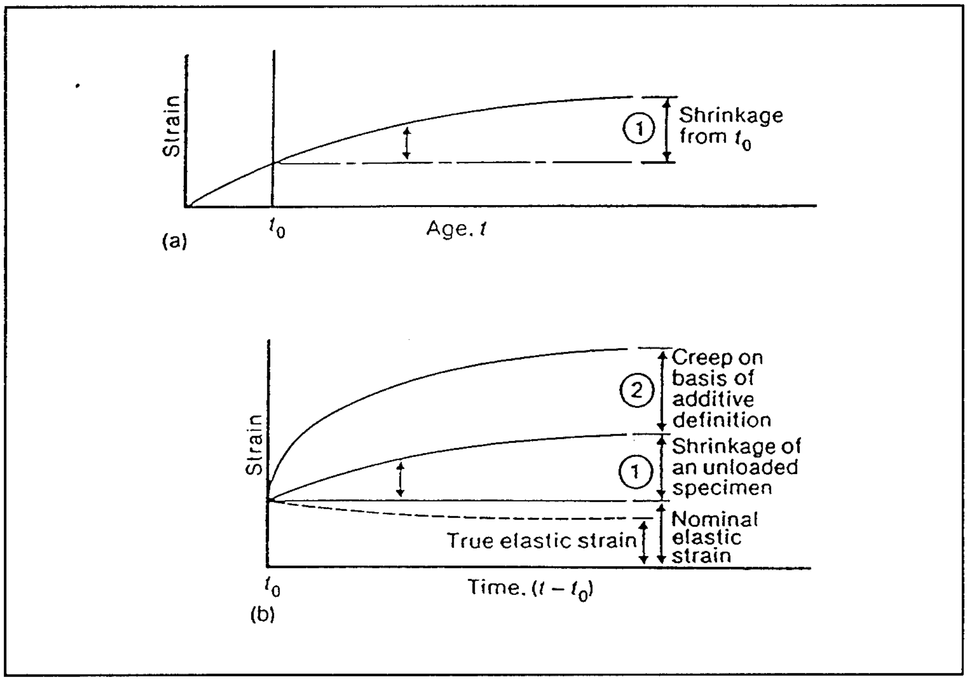



Figure 7. Schematic representation of creep and drying shrinkage (Neville, Dilger, and Brooks 1983)

## Shrinkage

Volume changes accompany the loss of moisture from fresh or hardened cementitious materials. The term "drying shrinkage" is generally used for hardened material. The term "plastic shrinkage" is used for unhardened concrete prior to time of setting, since its response to loss of moisture is quite different. "Carbonation shrinkage," which occurs when hydrated cement reacts with carbon dioxide from the atmosphere, can be regarded as a special case of drying shrinkage.

## Plastic shrinkage

Loss of water from fresh repairs, if not prevented, can cause cracking. The most common situation is surface cracking due to the evaporation of water from the surface. Suction of water from the repair by the substrate can also cause cracking or can add to the effects of surface evaporation. When the water is removed from the cementitious paste by evaporation, a complex series of menisci is formed. These, in turn, generate negative capillary pressures causing the volume of the paste to contract. The effects of plastic shrinkage are not uniform throughout the material and are restrained at the interface. Also, differential volume changes can cause cracking under induced tensile stresses.

Plastic shrinkage cracking is most common on the horizontal surfaces of repair where rapid evaporation occurs. Its occurrence affects the integrity of the repair and reduces its durability. Plastic shrinkage may be aggravated by special environmental conditions such as a combination of high wind velocity, low relative humidity, high air temperature, and a high temperature of the repair material. The most effective method to control plastic shrinkage is to ensure that the surface of the repair is kept moist until it has been finished.

Cracks caused by plastic shrinkage occur before time of setting of a cement-based material. During this time, the material is still in a semifluid or plastic state. The study of plastic shrinkage cracking is complicated because the material properties that determine whether such cracks will form are time-dependent and change rapidly during the first few hours in the life of the material. Such rapidly changing time-dependent properties include: the rate at which water is lost from the material in response to evaporative conditions, the degree to which the loss of water results in volume reduction, the consistency or stiffness of the mixture, and the development of the tensile stress and tensile strain capacity of the material. While the material is undergoing shrinkage due to the loss of water, it may be sufficiently fluid to comply with the volume change and, thus, develop relatively low tensile stresses. Alternately, the material may be so stiff as to resist the volume changes and thus develop relatively high tensile stresses as compared with the tensile capacity of the material at that time (Chatterji 1982).

As an example of this interaction in the material properties, it has been observed that a very fluid mixture (high water content), while having a potential for greater volumetric shrinkage than a stiffer mixture with lower water content, may in fact show little or no plastic shrinkage cracking because the fluid concrete remained sufficiently mobile to allow it to accommodate the volume change (Ravina and Shalon 1971).

Similarly, if the development of the tensile capacity of the material is more rapid than the development of shrinkage stress or strains, little or no cracking may occur. Ravina and Shalon (1960) noted less cracking in slabs cast in direct sunlight than for similar slabs cast in the shade and based their explanation of this observation on an acceleration in the rate of strength gain induced by thermal radiation. The warming of the slab surface may also have caused expansion that offset the shrinkage.

The shrinkage that is the root cause of these cracks is induced by the loss of water. It is commonly held that plastic shrinkage cracking develops when the rate of evaporation exceeds the rate at which bleed water rises to the surface. There is also a high probability of the formation of plastic shrinkage cracks when the rate of evaporation from the surface of the cement-based material is in excess of 0.2 $lb/ft^2/hr$ (1,000 $g/m^2/hr$) (American Concrete Institute (ACI) 1982). It should be noted that all considerations related to evaporation rate only refer to a surface completely covered by water. As soon as the surface water layer is gone, evaporation follows the laws of drying shrinkage.

It is clear that evaporation of water initiates the shrinkage that can ultimately lead to plastic cracking. On the basis of this study, the severity of cracking or whether cracks form at all depends on such factors as mixture proportions and construction operations. It has been observed that for given evaporative conditions, mixtures with a higher paste content have a higher tendency to crack. Similarly, it has been demonstrated that the formation of plastic shrinkage cracks is influenced significantly by finishing operations.

The initial shrinkage can be explained only *in part* by the loss of water. A shrinkage can, in fact, be equally observed in the absence of any evaporation. This shrinkage appears to be a settling by gravity and the action of capillary forces (Davis 1940). The first cause, however, is dominant, and by the time the surface becomes dry, the loss in volume is of the order of magnitude of the loss of the water up to the times when the grains enter into contact and a structure is established. Then, the law of shrinkage assumes a different form. It is observed that all bleeding of the water in part opposes the first shrinkage by producing a wet surface which prevents internal evaporation. The observation is made in various studies that cement paste protected against evaporation by an impermeable membrane shrinks due to internal desiccation of the mass by hydration of cement consuming the internal humidity. This phenomenon has a practical significance in respect to concrete and repair materials which are cured with the help of solid and liquid membranes. In the absence of replacement of that part of the free water which is chemically taken up for hydration, the material will tend to shrink, developing distress in the body of the concrete at an early age when it has very little strength to resist the stress.

A detailed review of literature in accordance with the scope of this study to determine the state of the art of factors affecting the shrinkage of concrete and other cement-based materials allows us to make the following conclusions:

*a.* The fundamental processes underlying shrinkage are yet to be fully understood.

*b.* Shrinkage is for the major part conditioned by evaporation. It is difficult to define the law of this evaporation in terms of time. However, it can be emphasized that shrinkage consists of three phases or periods. It begins with the wet surface where the speed of evaporation is constant. Then it diminishes until the concentration at the surface reaches a value in equilibrium with the air. Finally, in a third period, a movement or diffusion of the internal water toward the surface begins. It is the third phase (drying shrinkage) that becomes all-important for the solid cement-based material in which the surface water is in a relatively small quantity compared to the free water of the pores.

## Drying shrinkage

As previously indicated, drying shrinkage of hardened material is a much more important and critical phenomenon than plastic shrinkage. The definition of "drying shrinkage" is "shrinkage resulting from loss of moisture" (ACI 1993).

Neville (1981) defines drying shrinkage as the volume change associated with the loss of water from hardened concrete in unsaturated air. As shown in Figure 8, part of the initial volume change is irreversible and must be distinguished from the reversible portion resulting from alternating wet and dry conditions (Soroka 1980). The main focus in this investigation is on the causes and effects of the initial drying rather than on the mechanics of the reversible movements.

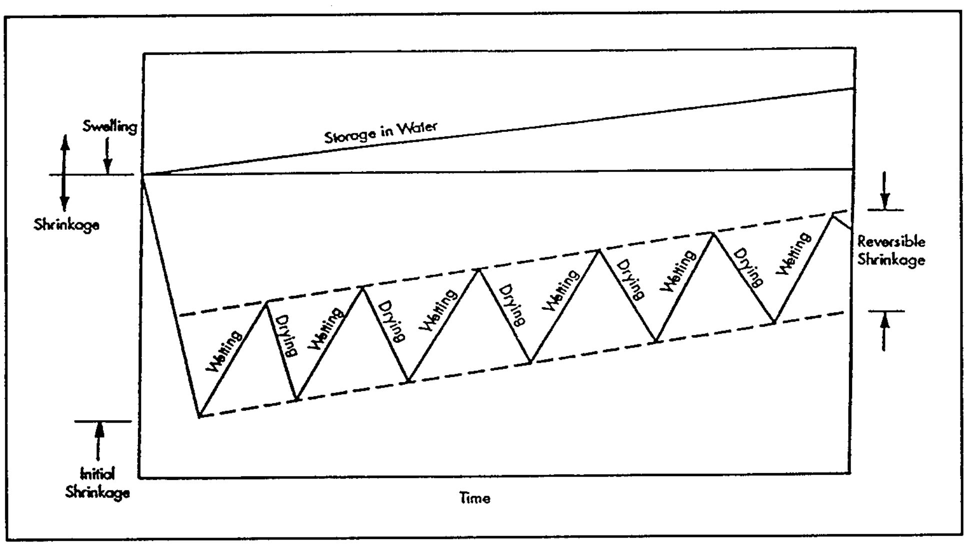



Figure 8. Schematic description of volume changes in cement paste due to alternate cycles of drying and wetting (Soroka 1980)

Washa (1956) defined drying shrinkage as "drying shrinkage of concrete is caused principally by the contraction of the calcium silicate gel...when the moisture content of the gel is decreased."

The basic mechanism of drying shrinkage of cement paste has been due to diffusion of capillary and absorbed water to the environment. This diffusion process, however, is influenced by the composition and density of the cement paste. It has also been shown (Powers 1959) that the smaller the amount of water-filled space per unit of cement (or the smaller the water to cement ratio) at the beginning, the less the proportion of capillary pores in the mature cement.

According to Ishai (1965), an increase in w/c ratio would intensify the shrinkage of cement paste and would accelerate the volume contraction process by providing more space for free-water diffusion. Further, the higher the percentage of capillaries and voids in the concrete system due to an increase in w/c ratio would reduce the rigidity of the solid matrix and its capacity to resist deformation. It is, therefore, expected that the shrinkage of cement paste will be greater if the w/c ratio is higher. The higher shrinkage of high-strength concrete can likely be attributed to the greater cement content, which is accompanied by a considerably greater amount of heat and, thus, rate of hydration.

The proportion of aggregate has an influence on the shrinkage of concrete since aggregate restrains the shrinkage of cement paste. It should be noted, however, that the results of study (Smadi, Slate, and Nilson 1987) clearly demonstrate that the w/c ratio discussion does not apply to high-strength concrete which has a greater rate of shrinkage than low-strength concrete and medium-strength concrete with the same w/c.

Powers (1959), Meininger (1966), and Tremper and Spellman (1963) point out that the water demand of the separate materials used in concrete is the major determinant of the shrinkage of concrete. Powers and Tremper and Spellman emphasized the cumulative effect on shrinkage in making poor choices in the selection of materials. Powers' shrinkage results, clarified by the Committee on Durability of Concrete, Physical Aspects-Drying Shrinkage (1963, 1964) in the form shown in Table 3, show the individual and cumulative effects of the most unfavorable versus the most favorable material choices with regard to six factors influencing the amount of shrinkage.

Powers assumed a constant w/c ratio and concluded: "Wrong choices of alternatives (with respect to volume change) can result in about seven times as much shrinkage as would result from the best choices."

Ytterberg (1987) summarized the results of Meininger's (1966) paper in Table 4.

**Table 3**
**Individual and Cumulative Effects of Various Factors in Concrete Shrinkage (Powers 1959)**

| Factor | | Effect[1] | |
|---|---|---|---|
| **Favorable** | **Unfavorable** | **Individual** | **Cumulative** |
| Cement of optimum $SO_3$ | $SO_3$ deficiency | 1.5 | 1.5 |
| Cement with 15% retained on No. 200 | 0% retained on No. 200 | 1.25 | 1.9 |
| Less compressible aggregate (quartz) | More compressible (Elgin gravel) | 1.25 | 2.4 |
| More aggregate (1-1/2-in. max. size) | Less aggregate (1/4-in. max. size) | 1.3 | 3.1 |
| More aggregate (stiff mixture) | Less aggregate (wet mixture) | 1.2 | 3.7 |
| No clay in aggregate | Much bad clay in aggregate | 2.0 | 7.4 |

[1] Multiplication factor for potential increase in shrinkage.

**Table 4**
**Effect of Various Factors on Concrete Shrinkage (Ytterberg 1987)**

| Factors | Max. Effect (%) |
|---|---|
| Coarse aggregate source effect | 100 |
| Fine aggregate source effect | 20 |
| Total aggregate source effect | 150 |
| Washing out minus No. 200 mesh | 15 |
| 2-1/2- vs 3/8-in. max. aggregate size | 25 |
| Fine aggregate grading from coarse to fine | 0 |
| Cement source | 15 |
| Cement factor | 10 |
| Slump | 5 |
| Curing: 7 days vs 3 days | 5 |

Regarding these variables and their effect on shrinkage, Meininger concluded that due to the source of coarse aggregate alone, concrete shrinkage can vary up to 100 percent.

Meininger's test results on the variation of shrinkage as related to the type of coarse aggregate are shown in Table 5. Fine aggregate can have an important effect (up to 20 percent) on concrete shrinkage. A change in both coarse aggregate and fine aggregate increased shrinkage as much as 150 percent over a control concrete. Shrinkage of concrete made with "as received" aggregates was up to 20 percent greater than with washed aggregates.

**Table 5**
**Effect of Coarse Aggregate on Drying Shrinkage of Concrete[1] (Ytterberg 1987)**

| Coarse Aggregates Rock Types | Shrinkage, millionths[2] 7 days | 182 days |
|---|---|---|
| Quartz | 180 | 530 |
| Igneous, andesite, sandstone | 180 | 560 |
| Greywacke, quartz, limestone, granite | 200 | 620 |
| Granite, quartzite | 220 | 640 |
| Schist, granite genesis | 210 | 660 |
| Impure limestone, sandstone, igneous | 230 | 640 |
| Igneous | 210 | 700 |
| Sandstone, limestone | 240 | 700 |
| Granite, granite gneiss | 240 | 750 |
| Sandstone | 230 | 740 |
| Sandstone, greywacke | 290 | 920 |
| Sandstone, greywacke | 300 | 900 |
| Sandstone. greywacke | 320 | 900 |

[1] 3- × 4- × 16-in. (76- × 102- × 406-mm) prisms dried at about 70 °F (21 °C) and 50% RH. Stock used in all.
[2] Millionths inch/inch.

The grading, composition, and the physical and mechanical properties of the aggregate have an important effect on concrete shrinkage, because aggregate particles embedded in cement paste restrain drying shrinkage (Table 6). Well-graded aggregates with a large maximum size have a low void space and, consequently, require a relatively small amount of paste. Larger maximum sizes of aggregates are effective in reducing shrinkage. Concrete of the same cement content and slump containing 3/8-in. maximum size aggregate usually develop from 10 percent to 20 percent greater drying shrinkage than concrete containing 3/4-in. maximum size aggregate, and from 20 percent to 35 percent greater drying shrinkage than concrete containing 1-1/2-in. maximum size aggregate. The actual amounts are dependent on variables such as aggregate type, length of air-drying period, cement content, and test procedure details.

The mechanical properties of the aggregate are influential in the behavior of concrete in shrinkage. The theory of Pickett (1956) predicts the influence of the elastic modulus and Poisson ratio of the aggregate in determining the degree of restraint of the aggregate on the shrinking paste.

In addition to the previously mentioned influence of the mechanical properties of the aggregate on the shrinkage of concrete, a few aggregates undergo excessive volume changes themselves on wetting and drying and,

**Table 6**
**Cumulative Effect of Adverse Factors on Concrete Shrinkage (Tremper and Spellman 1963)**

| Effect of Departing from Use of Best Materials and Workmanship | Equivalent Increase in Shrinkage, % | Cumulative Effect |
|---|---|---|
| Temperature of concrete at discharge allowed to reach 80 °F, whereas with reasonable precautions temperature of 60 °F could have been maintained. | 8 | 1.00 x 1.08 = 1.08 |
| Used 6- to 7-in. slump where 3- to 4-in. could have been used. | 10 | 1.08 x 1.10 = 1.19 |
| Excessive haul in transit mixture, too long a waiting period at job site, or too many revolutions at mixing speed. | 10 | 1.19 x 1.10 = 1.31 |
| Use of 3/4-in. maximum size aggregate under conditions where 1-1/2 in. could have been used. | 25 | 1.31 x 1.25 = 1.64 |
| Use of cement having relatively high shrinkage characteristics. | 25 | 1.64 x 1.25 = 2.05 |
| Excessive "dirt" in aggregate due to insufficient washing or contamination during handling. | 25 | 2.05 x 1.25 = 2.56 |
| Use of aggregates of poor inherent quality with respect to shrinkage. | 50 | 2.56 x 1.50 = 3.84 |
| Use of admixture that products high shrinkage. | 30 | 3.84 x 1.30 = 5.00 |
| Total Increase | Summation 183% | Cumulative 40 |

aggregate that is shrinking fails to restrain the shrinking of the paste (Roper, Cox, and Erlin 1964).

The part played by the composition of the concrete is very important in creating shrinkage. First of all, the nature of the cement has a considerable influence. Shrinkages increase in the following order: ordinary portland, aluminous cement, slag, and portland with high early strength (L'Hermite 1960). In each category, they are extremely variable. According to L'Hermite (1947), an ordinary portland cement gives 0.22 percent shrinkage in a neat paste at 1,000 days and a portland having a high early strength gives 0.35 percent shrinkage. Dutron (1934) indicates for ordinary portland a figure of the same order of magnitude, for aluminous cement, 0.25 percent. The same author shows that in portland and slag mixtures, the shrinkage increases with the proportion of the latter. Figure 9 gives a number of average curves relating to cements manufactured in France (Dutron 1934).

Graf (1933) was the first to express concern with the influence of the grain size of the cement on shrinkage by bringing out the increase of the latter when the specific surface increases. Haller (1940) has shown that shrinkage increases from 0.117 percent to 0.169 percent in 90 days when the specific surface increases from 1,355 $cm^2/g$ to 2,280 $cm^2/g$ for the same portland cement. Lafuma (1956) brings out a variation of the same kind on mortar: from 2,000 to 5,000 $cm^2/g$ obtained with the same clinker, the shrinkage increases from 0.05 percent to 0.17 percent, while the losses in weight are

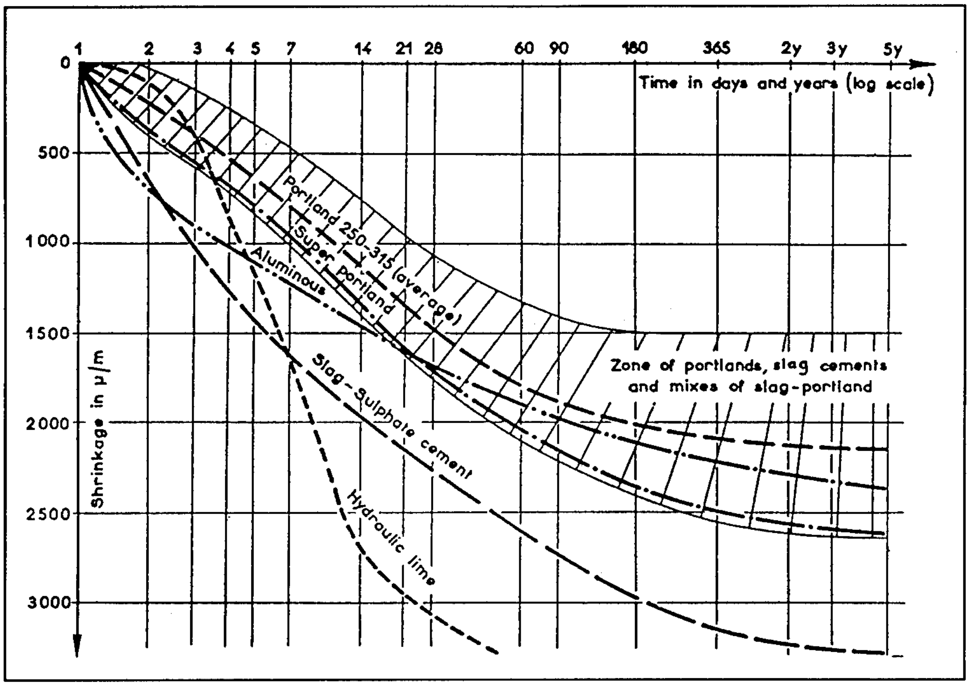



Figure 9. Shrinkage of some cements in pure paste (Dutron 1934)

less for the highest specific surfaces. The latter fact confirms what we had already established, that with an identical quantity of evaporated water, shrinkage increases with the volume of the hydrated paste, and this volume does increase with the specific surface.

Frequently it is assumed that high-range water reducers (HRWR) or superplastisizers will reduce shrinkage in proportion to their ability to reduce water, but references cited below indicate otherwise.

Table 7 shows that when compared to an extremely low slump control, concrete made with HRWR and containing 10 percent to 20 percent less water had only slight reductions in shrinkage (Whiting 1979).

Rixom and Waddicor (1981) show little difference in shrinkage between a control and a concrete containing HRWR. They found that the shrinkage of concrete made with normal dosage rates of melamine and naphthalene HRWR was greater and less, respectively, than the shrinkage of a control concrete with a 0.65 w/c. At double dosage rates, they found that shrinkage of concrete made with both types of HRWR was greater than the shrinkage of the control concrete.

The size and shape of the concrete repair have a considerable effect on the rate and the total amount of shrinkage. In large repairs, differential volume changes occur with the largest shrinkages found at and near the surface. Because of the large moisture variations from center to surface, tensile stresses are set up at the end and near the surface, while compressive stresses

**Table 7**
**Effect of High-Range Water Reducers on Drying Shrinkage (Whiting 1979)**

| Cement | Normal Cement Content (lb/cu yd) | Admixture | Water Content (lb/cu yd) | Net W/C Ratio | Net Air (%) | Slump (in.) | Water reduction (%) | 7 d | 28 d | 3 mo | 6 mo | 9 mo |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 21734 Type I | 376 | None | 264 | 0.7 | 2 | 2.2 | | 0.02 | 0.04 | 0.06 | 0.06 | 0.06 |
| | | Mighty 150 | 230 | 0.61 | 2.2 | 2.1 | 12.7 | 0.03 | 0.04 | 0.05 | 0.05 | 0.05 |
| | | Melment | 239 | 0.63 | 1.7 | 1.7 | 9.6 | 0.02 | 0.04 | 0.05 | 0.05 | 0.05 |
| | | Lomar-D | 238 | 0.62 | 2.9 | 2 | 10 | 0.02 | 0.04 | 0.06 | 0.06 | 0.05 |
| | | FX-032C | 219 | 0.59 | 4.6 | 2.1 | 16.9 | 0.02 | 0.05 | 0.05 | 0.05 | 0.06 |
| 21734 Type I | 517 | None | 258 | 0.49 | 2 | 2.8 | | 0.03 | 0.04 | 0.06 | 0.06 | 0.06 |
| | | Mighty 150 | 220 | 0.43 | 2.7 | 2.7 | 14.7 | 0.03 | 0.04 | 0.06 | 0.06 | 0.05 |
| | | Melment | 222 | 0.42 | 1.8 | 2.5 | 13.7 | 0.02 | 0.04 | 0.04 | 0.05 | 0.05 |
| | | Lomar-D | 217 | 0.41 | 2 | 2.4 | 15.9 | 0.03 | 0.04 | 0.05 | 0.06 | 0.05 |
| | | FX-32C | 206 | 0.4 | 4.7 | 2.7 | 20.1 | 0.02 | 0.04 | 0.05 | 0.05 | 0.06 |
| 21734 Type I | 658 | None | 268 | 0.04 | 1.6 | 2.8 | | 0.03 | 0.04 | 0.06 | 0.06 | 0.06 |
| | | Mighty 150 | 215 | 0.32 | 2.2 | 3 | 19.7 | 0.03 | 0.04 | 0.05 | 0.06 | 0.05 |
| | | Melment | 220 | 0.33 | 1.8 | 2.2 | 18 | 0.02 | 0.04 | 0.04 | 0.05 | 0.05 |
| | | Lomar-D | 217 | 0.32 | 2 | 3 | 18.8 | 0.02 | 0.04 | 0.05 | 0.05 | 0.05 |
| | | FX-32C | 217 | 0.32 | 2.9 | 2.4 | 19.1 | 0.02 | 0.04 | 0.05 | 0.05 | 0.06 |
| 21736 Type I | 658 | None | 287 | 0.43 | 2.1 | 3.2 | | 0.03 | 0.05 | 0.06 | 0.07 | 0.07 |
| | | Mighty 150 | 237 | 0.35 | 2.5 | 2.7 | 17.3 | 0.03 | 0.05 | 0.06 | 0.06 | 0.06 |
| | | Melment | 243 | 0.36 | 2.2 | 1.7 | 15.3 | 0.02 | 0.04 | 0.05 | 0.05 | 0.06 |
| | | Lomar-D | 232 | 0.35 | 2.3 | 2.6 | 18.9 | 0.02 | 0.04 | 0.05 | 0.06 | 0.06 |
| | | FX-32C | 240 | 0.36 | 2.7 | 2.3 | 16.3 | 0.03 | 0.05 | 0.06 | 0.06 | 0.07 |

are developed in the interior. Hence, if the tensile stress near the surface is very high, surface cracks may appear. However, the action of creep may prevent cracking and may cause permanent elongation of the fibers in tension and shortening of the fibers in compression. The rate and ultimate shrinkage of a large mass of repair material are smaller than the values for small-size material specimens, although the action continues over a longer period for the large mass. The ultimate shrinkage of a large member, however, might not exceed two-thirds of that obtained for the smaller one.

Cement-based materials expand with a gain in moisture and contract with a loss in moisture. If kept continuously in water, they slowly expand for several years, but the total amount of expansion is normally relatively small and it is practically unimportant (usually less than 150 millionths). Material that is not continuously wet is subject to water loss with resulting shrinkage. Since material exposed to the atmosphere loses some of its original water, it normally exists in a somewhat contracted state compared to its original dimensions.

The amount of moisture in materials is affected by the relative humidity of the surrounding air. After the material has dried to a constant moisture content at one atmospheric condition, a decrease in humidity causes it to lose moisture or an increase causes it to gain. The cement paste and the material of which it is a part shrinks or swells with each such change in moisture content generating cycling of opposite sign stresses.

Alternate wetting and drying causes cycles of shrinking and swelling. The effects of these moisture movements are illustrated schematically in Figure 10 (Portland Cement Association (PCA) 1967).

Troxell, Raphael, and Davis (1958) have shown that although shrinkage may occur even after 28 years, the rate of shrinkage decreases rapidly with time. Figure 11 shows that about 20 percent of the 20-year shrinkage occurs in the first 14 days and 60 percent occurs during the first 90 days. While prolonged moist curing will delay the advent of shrinkage, the effect of moist curing on the magnitude of the ultimate shrinkage is small. The decreasing rate of shrinkage with time is corroborated by data collected from field and laboratory cured samples made from Texas aggregates (Ingram and Furr 1973). In this study, high-strength concrete using crushed limestone and siliceous river gravel were used to cast test specimens.

Shrinkage data were collected for about 500 days, and hyperbolic regression equations were developed for each series of tests, as shown in Table 8.

This work corroborates the work of other researchers. Most of the measured shrinkage occurs in the first few weeks. The values in the denominators of the regression equations shown in Table 8 indicate that the time, in days, for one-half the shrinkage to occur is between 10 and 25 days.

The average for all the mixtures tested was about 3 weeks. The mixtures used by Ingram and Furr (1973) had different cement factors, but research by the U.S. Bureau of Reclamation (USBR) has indicated that cement content has only a minor influence on shrinkage (USBR 1963). However, these data have been disputed by the work of Haller (1940) who shows an effect of w/c ratio, but the effect does not appear until after 28 days.

Emmons (1993) stated that about 70 percent of the ultimate drying shrinkage of surface repairs occurs in the first 30 days.

The study of length change of concrete patching materials (Alberta Transportation and Utilities 1992) indicated that for the specimens which exhibited shrinkage only, a majority of the shrinkage (greater than 50 percent) generally occurred during the initial 7 days after casting, and further concluded, "Although the test results indicated some shrinkage beyond 28 days, the rate of increase on length change is observed to decrease with time. Consequently, evaluation of the patching materials is not expected to be significantly affected if the products are evaluated at an age of 28 days versus 120 days, provided the maximum shrinkage limit is adjusted to allow for the earlier test age."

Shrinkage may continue for several years depending on the size and shape of the concrete member. The rate and ultimate amount of shrinkage are smaller for large masses of concrete than for smaller masses, although shrinkage continues longer for the large mass.

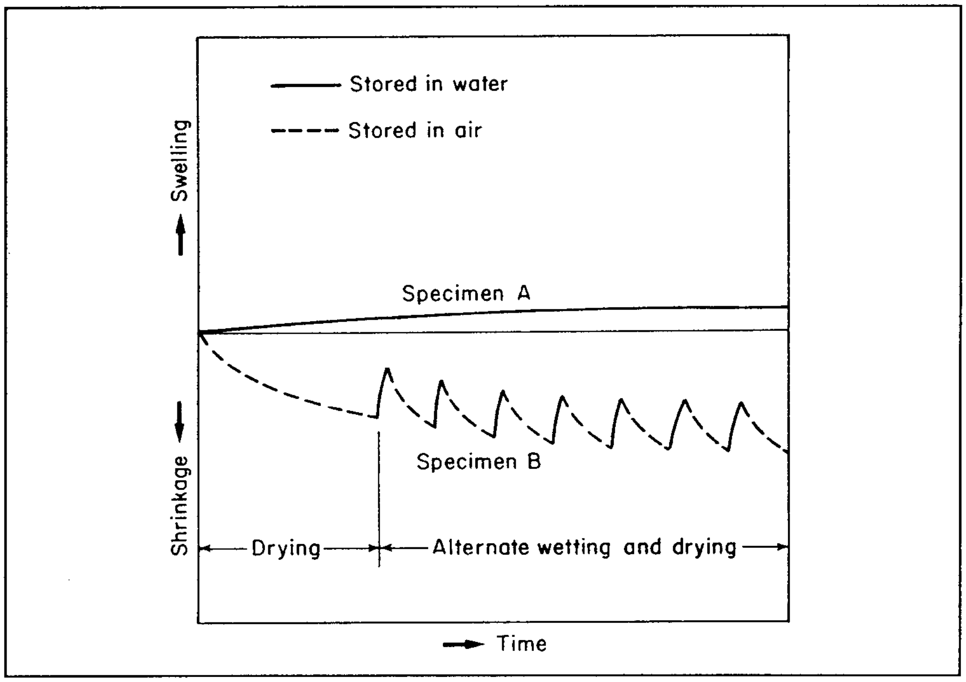



Figure 10. Schematic illustration of moisture movements in concrete (PCA 1967)

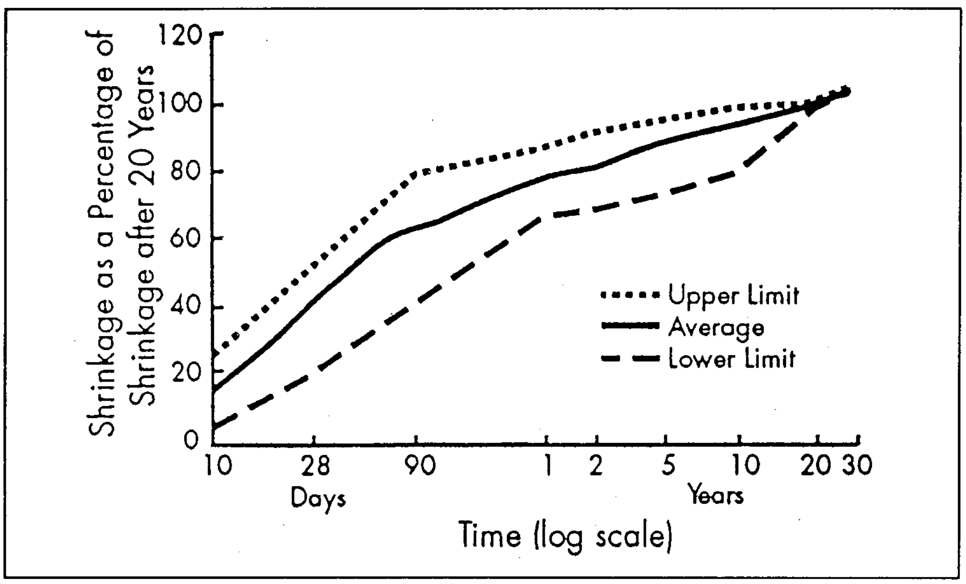



Figure 11. Range of shrinkage-time curves for different concretes stored at relative humidities of 50 percent and 70 percent (Troxell, Raphael, and Davis 1958)

**Table 8**
**Total Shrinkage at 500 Days and Estimated Shrinkage-Time Functions for Field Cured Specimens (Ingram and Furr 1973)**

| Size | Type | Shrinkage ($10^{-6}$in./in.) | Function |
|---|---|---|---|
| Fine<br>Coarse | Limestone and Siliceous Sand<br>Crushed Limestone | 300 | \F(315T, 20+T) |
| Fine<br>Coarse | Limestone and Siliceous Sand<br>Siliceous and Limestone Gravel | 510 | \F(525T, 10+T) |
| Fine<br>Coarse | 70% Limestone Sand/30% Siliceous Sand<br>Limestone Gravel | 360 | \F(380T, 25+T) |
| Fine<br>Coarse | Siliceous Sand<br>Crushed Limestone | 280 | \F(290T, 25+T) |

The shape of a structural member has an influence on the amount of drying shrinkage. Recent tests tend to indicate a surface volume-to-shrinkage relationship that can be expressed as a volume-to-surface ratio. The degree of correlation found between volume-to-surface ratios and shrinkage and creep (see following section) is satisfactory for purposes of practical design.

To summarize our studies of shrinkage presented in this report, it should be indicated that drying shrinkage of cement-based material is caused primarily by the contraction of the calcium silicate gel in the hardened cement-water paste when the moisture content of the gel is decreased. Among the more important factors that influence drying shrinkage are the unit water content of the material, the characteristics and amounts of admixtures used, the proportions of the mixture, the mineral composition of the aggregate, the size and shape of the mass, the amount and distribution of reinforcing steel, the curing conditions, the length of the drying period, and the humidity of the surrounding air.

Shrinkage of repair materials depends on many variables which are interdependent such as condition of the interface (how dry it is, how absorbent). Because of the complex nature of the physical and chemical factors in the shrinkage process and because of their coupling with each other, it is difficult to establish similitude which would account for variations in all affecting factors, such as the thickness and size of repair, sustained and repeated stresses, maximum size of aggregate, size and spacing of reinforcing steel, temperature, and humidity levels, etc.

## Modulus of Elasticity

The modulus of elasticity of repair material is important in determining the stress regime in the repair system at the interface (transition zone) between the repair and the existing concrete substrate. The derivations of Timoshenko

(1925) show that differences in modulus between the layers have a significant effect on the thermally induced stresses.

The factors affecting the modulus of elasticity are related to compressive strength and density. Thus, factors that affect strength should similarly influence modulus. The dominant parameter is, of course, the porosity. As the w/c ratio is increased, the modulus will decrease markedly.

As was noted previously, the factors influencing the modulus of cement-based materials essentially include all the factors affecting strength: w/c, aggregate type and grading, curing conditions, and age at the time of testing.

However, one apparent inconsistency in the compressive strength-elastic modulus relationship is the moisture dependency. The strength of saturated concrete and other cement-based materials is lower than that of dry materials, while for elastic modulus, the reverse is true.

For a given cement-based material, the modulus of elasticity increases with age during hardening in accordance with a law that is approximately proportional to the square root of the compressive strength, and it is greatly influenced by the humidity of the air in which the specimens were stored. For example, for concrete containing 350 kg of cement per $m^3$ with siliceous aggregate, test results have shown the following moduli at 200 days (Table 9) (L'Hermite 1960).

**Table 9**
**Moduli of Elasticity of Concrete Cured at Different Relative Humidities (L'Hermite 1960)**

| Relative Humidity of Curing Medium % | Modulus of Easticity $kg/cm^2$ |
|---|---|
| 35 | 340,000 |
| 50 | 365,000 |
| 75 | 385,000 |
| 99 | 450,000 |
| Water | 450,000 |

The modulus of material is substantially affected by the type and the amount of the aggregate. Effect of aggregate on the modulus of elasticity of concrete is demonstrated in Figure 12 (Neville 1981) and Figure 13 (Lundy, McCullough, and Fowler 1990), and modulus of concrete and its components are shown in Table 10.

Since the value of the elastic modulus is partly dependent on microcracking at the matrix-aggregate interface caused by elastic mismatch between aggregate and cement matrix, the shape, texture, and total amount of aggregate will influence its value.

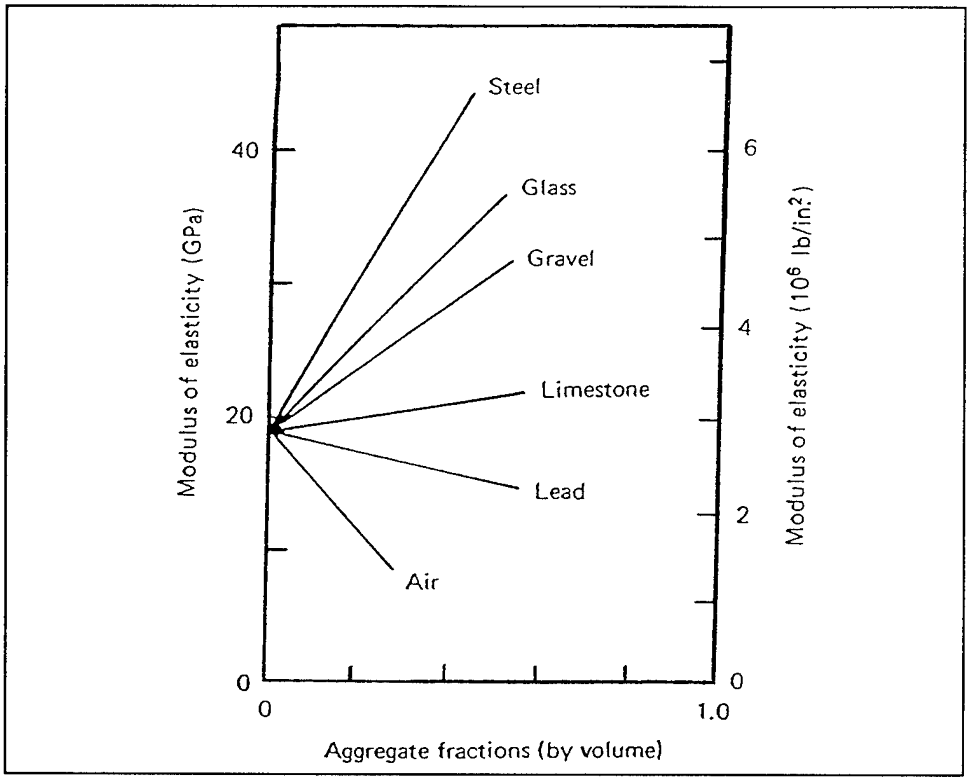



Figure 12. Effect of aggregates on the modulus of elasticity of concrete (Neville 1981)

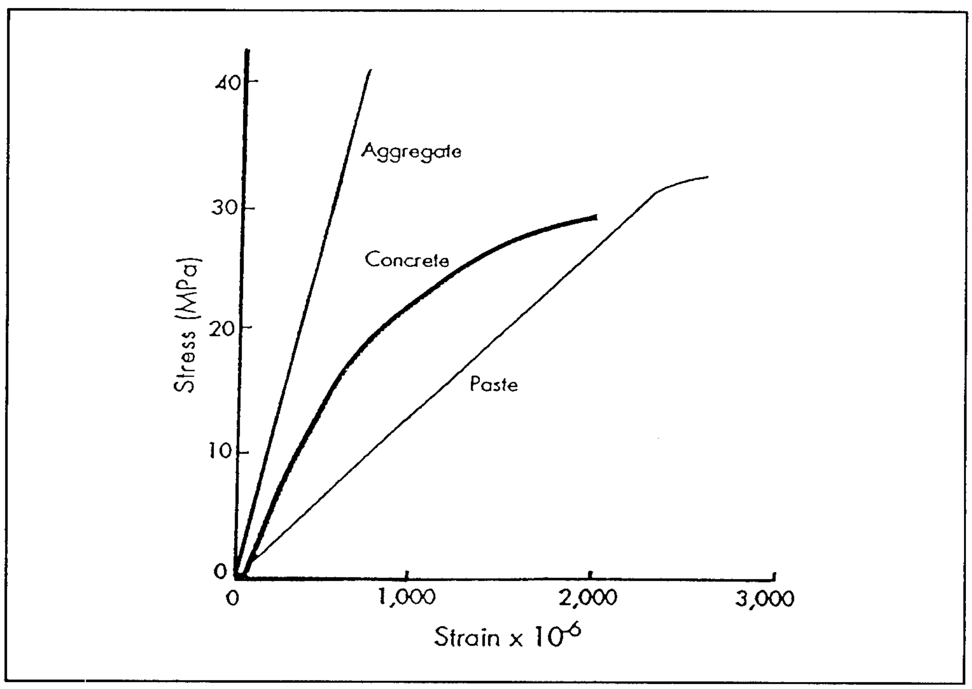



Figure 13. Stress-strain relationships from cement paste, aggregate, and concrete (Lundy, McCullough, and Fowler 1990)

An increase in modulus can also be expected from a decrease in the w/c. The relationship between w/c and modulus for several different aggregate types can be seen in Figure 14. The ranges of elastic modulus for concrete and its components are shown in Table 10 (Mindess and Young 1981).

The rate of moduli of elasticity gain with time is demonstrated in Table 11.

It has been established that internal cracks and flaws exist in a repair composite system for reasons other than service loads. As mentioned earlier, some of these are caused not only by the drying shrinkage of the repair material and its different coefficient of thermal expansion from the substrate

**Table 10**
**Moduli of Elasticity for Concrete and Its Components, GPa (lb/in$^2$) (Mindess and Young 1981)**

| | Normal Weight | Lightweight |
|---|---|---|
| Aggregate | 70-140 (10-20 x $10^6$) | 14-35 (2-5 x $10^6$) |
| Cement paste | 7-28 (1-4 x $10^6$) | 7-28 (1-4 x $10^6$) |
| Concrete | 14-42 (2-6 x $10^6$) | 10-18 (1.5-2.5 x $10^6$) |

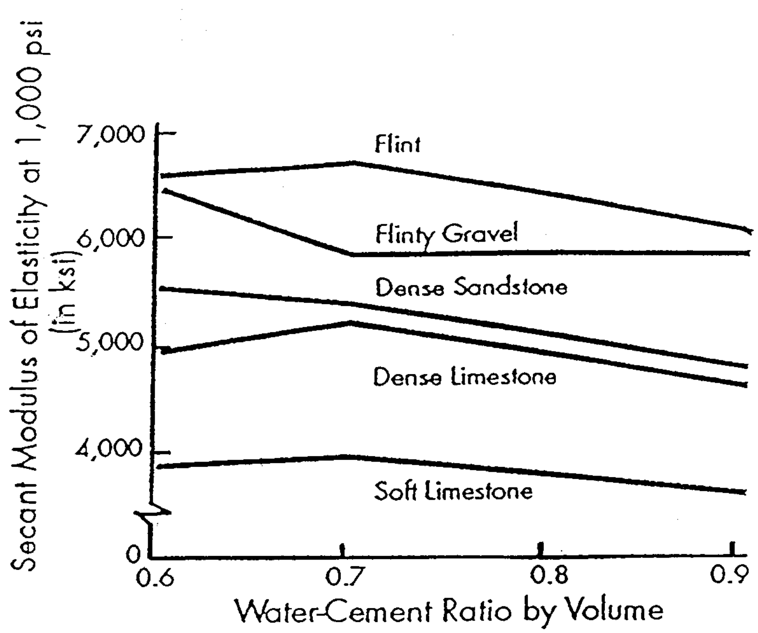



Figure 14. Effect of w/c ratio and type of aggregate upon modulus of elasticity. Mixtures contained six sacks of cement per cubic yard. Age of test was 56 days (Noble 1931)

**Table 11**
**Rate of Modulus Gain with Time (Lundy, McCullough, and Fowler 1990)**

| Time (days) | Percent of 28-day Modulus |
|---|---|
| 0.5 | 15 |
| 1 | 30 |
| 2 | 45 |
| 3 | 55 |
| 7 | 80 |
| 14 | 87 |
| 28 | 100 |

material but also to differences in the elastic moduli of the repair material and the substrate. The compatibility in elastic moduli, therefore, becomes, in some cases, an important factor because incompatibility may lead to considerable stress concentration when widely differential volume changes of the repair material in relation to the concrete substrate occur. Since, in such situations, the interfacial bond region (transition zone) is the weak link in the repair system, and cracks will tend to form in this region. In certain cases where the bond strength is high, cracks will occur in the matrix of the material which has the higher modulus of elasticity.

When external load is perpendicular to the bond line, as in the case of repaired pavement, differences in modulus of elasticity with regard to the varying stiffness between the repair materials and concrete substrate is not normally problematic.

In vertical repairs, however, where the service load is parallel to the bond line, differences in modulus of elasticity may cause load transfer to the high modulus material if the other materials yield under the stress. If the load transfer is beyond the load bearing capacity of the higher modulus material, it will fracture and damage the structure.

## Thermal Expansion

The coefficient of thermal expansion (CTE) is important to the successful analysis of stresses in the concrete repair system. The CTE, which gives a measure of dimensional contraction or expansion with changes in temperature, is an essential property of the composite repair system. When significant changes in temperature occur, a marked difference in the CTE will produce different volume changes between the repair material and the concrete substrate. Such differential volume changes may produce excessive stresses at the interface between the repair material and the concrete which causes bond failure or, in the case of high bond strength, failure within the lower strength material.

Thermal expansion of some rock types (Venecanin 1990) are shown in Table 12.

**Table 12**
**Coefficients of Thermal Expansion of Some Rock Types (Venecanin 1990)**

| Rock Type | Coefficient of Thermal Expansion<br>$10^{-6}$ per K | Coefficient of Linear Expansion<br>$10^{-6}$ per degree F |
|---|---|---|
| Limestone | -5.2 to 12.2 | -2.9 to 6.8 |
| Marble | -2.2 to 16.0 | -1.2 to 8.9 |
| Granite | 1.8 to 11.9 | 1.0 to 6.6 |
| Gabro, basalt, diabase | 3.6 to 9.7 | 2.0 to 5.4 |
| Diorite, andesite | 4.1 to 10.3 | 2.3 to 5.7 |
| Sandstone | 4.3 to 13.9 | 2.4 to 7.7 |
| Dolomite | 6.7 to 8.6 | 3.7 to 4.8 |
| Quartzite | 7.0 to 13.1 | 3.9 to 7.3 |
| Chert | 7.4 to 13.1 | 4.1 to 7.3 |
| Hardened cement paste | 9.0 to 25.4 | 5.0 to 14.1 |

The greater the difference between CTE of aggregate and hardened cement paste, the greater the reduction of mechanical properties and durability of concrete if it is exposed to temperature changes. Thermal properties of concrete constituents are shown in Table 13.

When concrete is exposed to temperature changes, unequal volume changes of its components cause tensile stresses and cracking of concrete and reduce its durability. This phenomenon is called thermal incompatibility of concrete components (TICC). In practice, TICC appears if aggregate in concrete has a very low CTE. Some limestones used as aggregate in concrete have significantly lower CTE. The greater the differences between the CTE of aggregate and hardened cement paste, the greater the reduction of mechanical properties and durability of concrete if it is exposed to temperature changes. Pearson (1941) presented cases of damage of concrete due to the TICC.

Many of the same factors affecting modulus of elasticity influence the coefficient of thermal expansion of cement-based materials, but the age of the material is not an important factor (Aslam, Saraf, Carrasquillo, and McCullough 1987).

**Table 13**
**Thermal Coefficient of Expansion of Concrete Components (Neville 1981)**

| Concrete Components | $10^{-6}/°C$ | $10^{-6}/°F$ |
|---|---|---|
| Cement Paste (Saturated) | | |
| w/c = 0.4 | 18-20 | 10-11 |
| w/c = 0.5 | 18-20 | 10-11 |
| w/c = 0.6 | 18-20 | 10-11 |
| Aggregates: | | |
| Granite | 7-9 | 4-5 |
| Limestone | 6 | 3.3 |
| Sandstone | 11-12 | 6.1-6.7 |
| Quartzite | 11-13 | 6.1-7.2 |
| Dolomite | 7-10 | 4-5.5 |
| Water | - | - |
| Air | - | - |
| Steel | 11-12 | 6.1-6.7 |
| Concrete | 7.4-13 | 4.1-7.3 |

The type of aggregate used in the mixture, however, does have a significant effect on the thermal coefficient. The linear coefficient of thermal expansion for neat cement paste varies from 6 to 12 millionths per degree Fahrenheit (Troxell, Davis, and Kelly 1968). The normal range for air-cured concrete is 4 to 6 millionths per degree Fahrenheit depending on the aggregate type. The reduction in the coefficient results from the restraining effect of the aggregate: as the paste tends to contract, the aggregate is put into compression, and this results in less total movement. Coefficients of thermal expansion of concrete made with different aggregates are shown in Table 14 (Neville 1981).

**Table 14**
**Coefficient of Thermal Expansion of 1:6 Concretes Made with Different Aggregates (Neville 1981)**

| Type of Aggregate | Air-cured Concrete $10^{-6}$ per °F | Water-cured Concrete $10^{-6}$ per °F | Air-cured and Wetted Concrete $10^{-6}$ per °F |
|---|---|---|---|
| Gravel | 7.3 | 6.8 | 6.5 |
| Granite | 5.3 | 4.8 | 4.3 |
| Quartzite | 7.1 | 6.8 | 6.5 |
| Dolerite | 5.3 | 4.7 | 4.4 |
| Sandstone | 6.5 | 5.6 | 4.8 |
| Limestone | 4.1 | 3.4 | 3.3 |
| Portland Stone | 4.1 | 3.4 | 3.6 |
| Blast Furnace Slag | 5.9 | 5.1 | 4.9 |
| Foamed Slag | 6.7 | 5.1 | 4.7 |

Coefficients are given for air-dried, water-cured, and air-cured and wetted concrete to emphasize the influence of moist curing on the coefficient of thermal expansion.

Table 14 shows that for all aggregate types, the coefficient of linear expansion decreases with increasing exposure to moisture during curing. The influence of the moisture condition applies mainly to the paste because the total movement is made up of two components - kinetic movement and swelling pressure. As the temperature increases, the surface tension of the pore water decreases and swelling takes place (Powers and Brownyard 1947). Obviously, the swelling pressure will not occur if the concrete is saturated or dry. Therefore, for the cement paste, the "kinetic" coefficient of thermal expansion is about 6 millionths, but the measured coefficient at normal humidity would be almost 12 millionths.

The factors discussed above help explain the variability of coefficients for the different moisture conditions. However, at early ages, if adequate curing has been provided, the relative humidity of the concrete will be near 100 percent. Therefore, the influence of moisture content on the thermal coefficient may be neglected.

Other factors also affect the thermal coefficient of the paste. However, Neville (1981) states that the chemical composition, fineness of the cement, and air-void content do not affect the thermal coefficient.

The thermal coefficient is important because it is used to estimate the movement associated with a given change in temperature. Thermal strain, $\epsilon$, is defined by:

$$\epsilon = aT\Delta$$

where

$a$ = the thermal coefficient

$\Delta T$ = the change in temperature from some reference condition

The determination of the strain implies not only that the thermal coefficient and the temperature at the time in question are known, but also that some reference temperature is known. This reference temperature is commonly referenced to as a set or curing temperature. The curing temperature is essential to the calculation of the thermally induced stresses and forms the reference point from which the temperature differential, $\Delta T$, is calculated.

Overall, the results of our study indicate resinous repair materials have significantly higher CTE's compared to nonresinous materials. Nonresinous materials tend to have values about equal to that of many unmodified concretes, and the addition of polymers to unmodified materials has little effect on their coefficients of thermal expansion.

Numerous examples in literature convincingly demonstrate that thermal compatibility is a very critical property to consider when specifying and selecting repair material, regardless of its chemical composition. The following example clearly demonstrates this fact. Epoxy resins exhibit CTE up to eight times greater than concrete. This gap in thermal compatibility is reduced by the addition of aggregates in the epoxy mortar. Such mortars, when passing the ASTM C 884 (ASTM 1994t) test can be specified and successfully used for the repair of structures with relatively high temperature changes. Unfortunately, such a generalized recommendation is not valid for all practical applications. Bridge and parking deck slabs employ horizontal repair techniques in which the forces of gravity allow the use of low viscosity epoxy mortars with high aggregate content. Thermal compatibility in such applications is possible to achieve, and this is the type of application where epoxy mortar may be specified. In cases of repairs involving overhead and vertical applications without formwork, gravity forces work against us. In these cases, only thixotropic (nonsag or gel) epoxy resins with limited aggregate content can be used. However, they are thermally incompatible with concrete and should not be specified for repairs of structures with relatively high temperature changes.

Sprinkel (1983) reported that the temperature changes to which bridge decks are typically subjected are sufficient to cause deterioration and eventual failure of polymer-concrete overlays. Deterioration is caused by the development of stresses in the bond between the concrete and the overlay. The stresses are the result of differences in the moduli of elasticity and the CTE's in the two materials. Thermally induced cracks have been noted in the overlay, the base concrete, and the bond interface - the majority of which cannot very well withstand stress. Cracks in the overlay increase its permeability, and cracks in the base concrete or the bond interface lead to delamination of the overlays. Sprinkel reported that failure can be grouped into three basic types, as follows:

*a.* The formation of vertical cracks through the thickness of the overlay.

The formation of vertical cracks increases the permeability of the overlay and reduces its effectiveness in preventing the infiltration of chlorides. It will be the predominant mode of failure on bridges where the shear strength of the base concrete and the bond strength are high or the modulus of elasticity of the overlay is high, or the tensile strength is low. Failure will likely occur after a few cycles of temperature change. The overlay will likely remain bonded to the base concrete until the freezing and thawing action cause delamination.

*b.* The shearing of portland cement concrete below the bond line.

Shearing of the concrete below the bond line causes the overlay to delaminate with concrete remaining bonded to its underside. Failure is most likely to occur when the shear strength of the base concrete is low, the bond is good and the tensile strength of the overlay is high.

Failure will likely occur after a few cycles of temperature change and will result in the delamination of the polymer-concrete overlay.

*c.* The delamination of the bond between the polymer-concrete (PC) overlay and the base concrete.

Delamination of the bond between the PC overlay and base concrete causes the overlay to delaminate with no concrete remaining on the underside. Failure is likely to occur when the surface preparation prior to the installation of the overlay is poor or when the shear strength of the base concrete and the tensile strength of the overlay are high. Where the initial bond is good, a significant number of thermal cycles may be required to complete the failure.

## Creep

The deformation of a material in response to load is known as *rheological behavior* (Philleo 1956). While instantaneous effects and time-dependent effects are not entirely separable, it is common to consider them separately as elastic properties (instantaneous) and creep (time-dependent).

A body that returns to its original dimensions after release of stress is elastic. Creep of concrete is considered to be an isolated rheological phenomenon associated with the gel structure of cement paste. Creep is a slow plastic deformation. When a cement-based material is loaded and remains under the influence of this load over a long period of time, it continues to become deformed.

It is commonly stated in studied literature that creep and shrinkage are interrelated phenomena because there are a number of similarities between the two. The Table 15 lists the various parameters that can be expected to affect creep and also drying shrinkage (Mindess and Young 1981).

Like shrinkage, creep is a cement-paste property with the aggregate acting as a restraint. The analysis of literature demonstrates that the process of creep is not clearly understood, and there is mounting evidence that creep and shrinkage occur by quite different mechanisms.

When concrete is loaded, the deformation caused by the load may be divided into two parts: a deformation which occurs immediately, and a time-dependent deformation which begins immediately but continues for years. The latter deformation is called "creep." Creep is defined in ACI 116R (ACI 1993) as "time-dependent deformation due to sustained load."

The first known study of creep is the one published by Woolson (1905). Troxell, Raphael, and Davis (1958) were the first to bring out the important influence of the humidity of the curing medium on creep.

**Table 15**
**Parameters Affecting Drying Shrinkage and Creep of Concrete (Mindess and Young 1981)**

| |
|---|
| Paste parameters |
| Porosity} w/c ratio and degree of hydration |
| Age of paste} w/c ratio and degree of hydration |
| Curing temperature |
| Cement composition |
| Moisture content |
| Admixture |
| Concrete parameters |
| Aggregate stiffness |
| Aggregate content (cement content) |
| Volume-to-surface ratio |
| Thickness |
| Environmental parameters |
| Applied stress} affects only creep |
| Duration of load} affects only creep |
| Relative humidity |
| Rate of drying |
| Time of drying |

The particular aspect of the gel structure of concrete which causes its unusual behavior is the accessibility of its large internal surface to water. In fact, Mullen and Dolch (1964) found no creep at all in oven-dried pastes. Mather (Committee on Durability of Concrete 1963 and 1964), in discussion of Mullen and Dolch (1964), noted that dry materials creep and that the failure to measure such simply meant that strains were below detection limit of gauges appropriate for wet concrete. The movement of water into and out of the gel in response to changes in ambient humidity produces the well-known shrinking and swelling behavior of concrete.

Numerous theories have been advanced to explain creep--thermally activated creep, distribution of absorbed water, and interlayer theory. There are elements of similarity among the three approaches, however, the different approaches treat the same phenomena in different ways and with different emphasis.

A principal view among investigators (Lyman 1934, Lea and Lee 1947, Hansen 1958) is that creep is closely related to shrinkage. In creep, gel water movement is caused by changes in applied pressure instead of differential hygrometric conditions between the concrete and its environment. This concept is supported by the similar manner in which creep and shrinkage curves are affected by such factors as w/c, mixture proportions, properties of aggregate, compaction, curing conditions, and degree of hydration.

Another explanation of the effect of gel water (Freyssinet 1951, Torroja and Paez 1954) is delayed elasticity. If a load is suddenly imposed on a body consisting of a solid elastic skeleton with its voids filled with a viscous fluid, the load will be carried initially by the fluid and will gradually be transferred to the skeleton as the fluid flows under load. This is the behavior exhibited by the rheological model known as Kelvin body, which consists of a spring and dashpot in parallel. The concept of delayed elasticity has been chiefly responsible for the widespread attempts to reproduce the rheological behavior of concrete by means of rheological models.

Creep has been attributed by some (Freudenthal 1950, Reiner 1960, Glanville and Thomas 1939) to the viscous flow of the cement paste. The reduction in the strain rate over time has been attributed by these investigators both to the increasing viscosity of the paste and to the gradual transfer of load from the cement paste to the aggregate. This concept is supported by the concept that creep strain is proportional to the applied stress over a wide range of stress. A convincing argument against this theory is the fact that the volume of concrete does not remain constant while it creeps. In fact, Poisson's ratio for creep has usually been less than the ratio for elastic stress. Another argument against the concept is the partial recovery of creep when the load is removed.

The fact that creep is associated primarily with the cement-paste phase of concrete produces a serious difficulty in the interpretation and application of creep data. Unless the work is restricted to very mature concrete, the specimens do not maintain constant physical properties throughout the test. Creep measurements must necessarily be made over a considerable amount of time and during that time the cement paste continues to hydrate. Frequently, information is desired at early ages when the cement is hydrating relatively rapidly.

A typical creep curve is shown in Figure 15. Within the normal stress ranges, creep is proportional to stress. The ultimate magnitude of creep of plain concrete per unit stress (pounds per square inch) can range from 0.2 to 2.0 millionths in terms of length but is ordinarily about 1.0 millionth or less. In the survey made by Smadi (1982), the load-induced, time-dependent deformations of concrete are largely attributed to the movement of capillary and absorbed water within the concrete system, to the movement of water in the environment, and to the development and propagation of internal microcracks. The rate and magnitude of creep strain associated with the first two processes would depend on the relative volume of pores and spaces in the cement gel and on the amount of water occupying these pores at the time of loading.

Troxell, Raphael, and Davis (1958) indicated the important influence of the humidity of the curing medium on creep. By the way of example, they indicated that after 1,600 days the following values for creep were obtained on a portland-cement concrete loaded at 28 days at 65 kg/cm$^2$ (Table 16).

Time-dependent deformations, which have a profound influence on the behavior of concrete repair systems, are some of the most certain and least understood phenomena in cement-based materials.

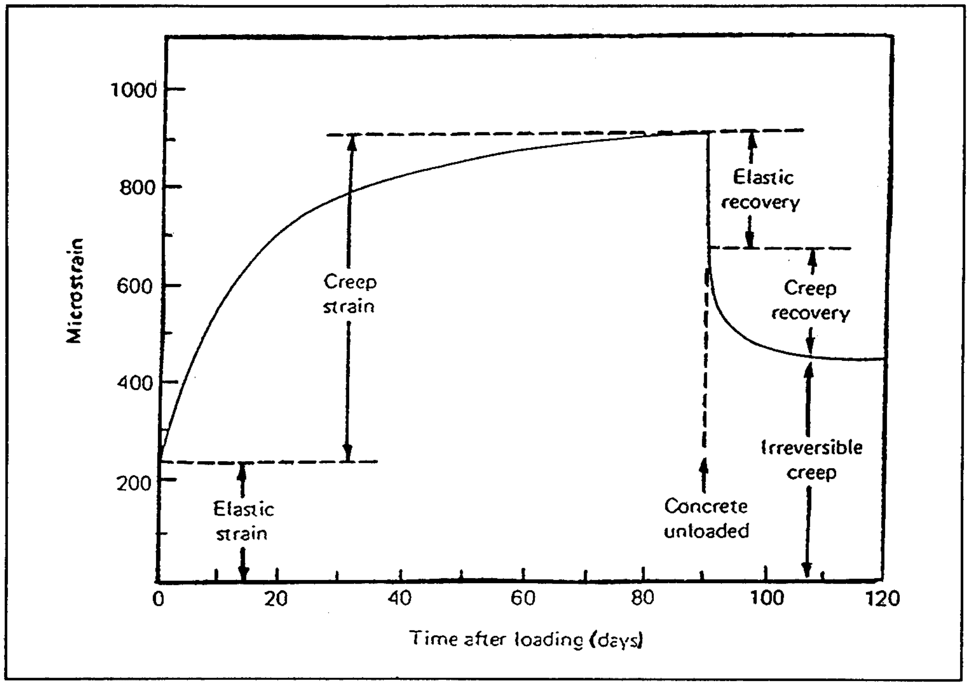



Figure 15. Typical creep curve for plain concrete (Mindess and Young 1981)

**Table 16**
**Influence of Humidity of the Curing Medium on Creep (Troxell, Raphael, and Davis 1958)**

| Relative Humidity | Total Creep $\mu$/m | Shrinkage without Load $\mu$/m | Calculated Creep $\mu$/m |
|---|---|---|---|
| Water.................... | 180 | -120 | 200 |
| Air 99 percent......... | 250 | -100 | 240 |
| Air 70 percent......... | 1,450 | +750 | 1,460 |
| Air 50 percent......... | 2,100 | +1,200 | 2,100 |

Analysis of literature indicates that shrinkage is an adverse property of cement-based materials. Creep can be a positive property--creep strains may heal cracks when stress relaxation occurs, and basic creep at very early stages may relieve stresses induced by restraint to differential thermal movement arising from the heat of hydration.

The requirements for structural and protective repairs differ considerably. The property of greatest significance is creep, particularly the variation of this property in different environmental conditions (Plum 1990a). For structural applications, low creep is desirable over the expected environmental range.

For protective applications, a higher creep is an advantage at some part of the environmental range.

The problem of *creep* performance in repair materials is a complex one. Many aspects of the subject need examination, such as: age at loading, duration of load, relationship between compressive, tensile, and flexural values, and the wide range of potential environments. Much of the available literature omits reference to creep, or makes only a passing mention, but sometimes its importance is discussed (Hugenschmidt 1982).

It is equally unfortunate that most manufacturers make no mention of this property in their literature and are unable to supply basic values or to advise on environmental effects.

Analysis of literature and personal communications demonstrate that attempts to establish the fundamental role of creep in relative volume changes of cement-based materials and in concrete repairs, in particular, are believed to be unjustified at this time in view of the continuing lack of a full understanding of the mechanism of creep.

It is clear that the great divide between structural and nonstructural repairs is governed by the property of creep. For structural repairs, creep must be carefully controlled to a value close to or less than that of the core concrete; otherwise, the repair efficiency decreases rapidly. For protective repairs, creep may be very useful to relieve any stresses resulting from external loading (Plum 1990a).

An analysis of literature during this study indicates that most creep-related publications have been concerned with creep in compression. Several investigators have studied creep in flexure because of the obvious application to beam deflections. A very limited amount of work has been done in tension (Ross 1954, McMillan 1915, Wajda and Holloway 1964).

Creep in tension is of considerable interest in estimating cracking potential due to stresses caused by moisture or thermal volume changes. Reduction of tensile stresses by tensile creep can minimize cracking. However, it is difficult to measure this property accurately because of the low tensile strength of cement-based materials. Therefore, it is not easy to draw quantitative comparisons between creep in compression and in tension.

It appears, however, that the initial rate of creep is higher in tension. Tensile creep, therefore, is greater for relatively short durations of load, although at longer times the reverse may hold. Flexural creep is complicated by the fact that part of the concrete is in compression and part in tension. Creep in the tensile and compressive fibers is not necessarily the same and can be affected to different extents by drying.

Tests on the flexural creep of polymer modified cement-based materials were reported by Plum (1990b). Tests were carried out on specimens of a cross section 25 by 5 mm (1 by 1/4 in.) deep over a span of 150 mm (6 in.). Curing was carried out at 35 °C (95 °F) and various humidities for 7 days followed by the creep test at a stress N/mm$^2$ for 14 days more. All materials tested showed a very marked increase in creep strain at high humidity.

Creep also occurs under dynamic loading, although it is difficult to separate those time-dependent strains due to creep and those due to progressive microcracking under the changing stress conditions (which leads to fatigue failure). It is generally found that dynamic creep is greater than static creep when placed under the same maximum stress. Creep strains appear to depend on the range of stress, the frequency of loading, and the duration of dynamic loading.

Also, some controversy exists concerning the mechanism of creep in cement-based materials. It is generally accepted that it is related to the movement of physically bound water in the gel structure. On this basis, the factors that affect compressive creep, viz., ambience, paste content, etc., should affect tensile creep in a similar manner. Powers (1966) has indicated that there can be no physical difference between the mechanisms of compressive and tensile creep. Bazant (1970) makes a restriction for the mechanisms to be the same, i.e., the level of stress be sufficiently small. Beyond the point of linearity between the applied stress and creep, microcracking occurs. In compressive creep, the point at which nonlinearity occurs between the applied stress and creep, in terms of the stress-strength ratio, varies from 0.30 to 0.75.

For the tensile loading, microcracking occurs at a stress-strength ratio of 0.20 or less. The structural behavior of concrete in tension is markedly different from its compressive behavior. Cook (1972a, b), and El Baroudy (1940) have shown that tensile creep increases as the volumetric aggregate content increases - a finding compatible with the microcrack propagation concept. By comparison, the reverse is true for compressive creep behavior, viz., that creep increases as the volumetric paste content increases.

The limited amount of publications on tensile creep indicate no consistent relationship between tensile creep and relative humidity. This is not the case, however, for compressive creep which decreases as the relative humidity of the environment increases. The publication by Cook (1972a) suggests that shrinkage-induced microcracking accounts for the inconsistent relationship between tensile creep and relative humidity.

Relief of tensile stresses in concrete caused by creep (Neville 1981) is shown in Figure 16.

Our study indicates that the majority of repair materials have an unjustifiably high compressive strength in excess of 6,000 psi (Table 17). In connection with this fact, it should be stated that if the material in a repair will be as strong as it can possibly become, it will have a high modulus of

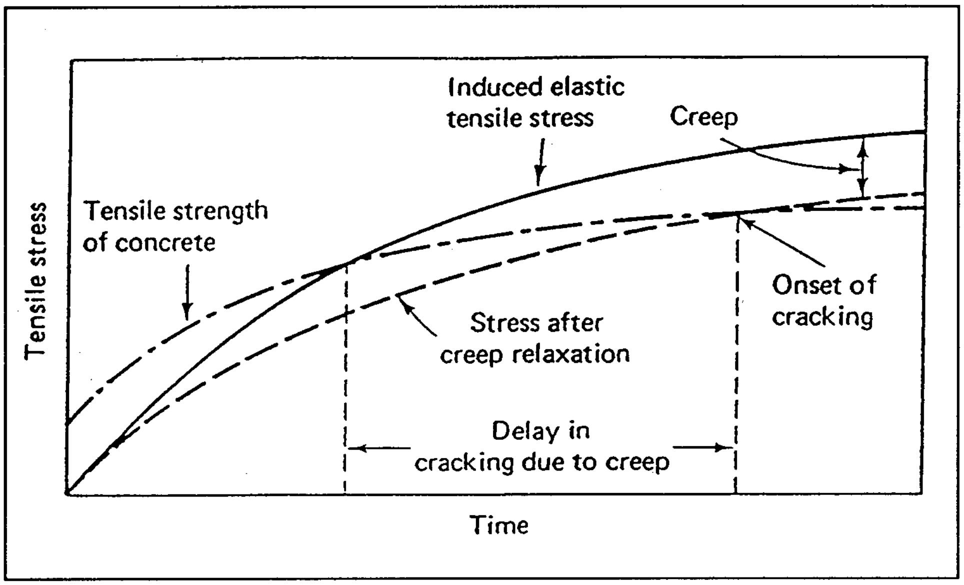



Figure 16. Relief of tensile stresses in concrete by creep (Neville 1981)

**Table 17**
**Compressive Strengths of Representative Surface Repair Materials**

| | |
|---|---|
| Cast-in-place | 7,000 psi |
| Trowel | 6,000 psi |
| Cast-in-place | 7,000 psi |
| Cast-in-place | 11,000 psi |
| Trowel | 6,950 psi |
| Cast-in-place | 6,100 psi |
| Rapid set | 8,500 psi |
| Shotcrete | 11,000 psi |

elasticity, be more brittle, and crack at a lower strain level. Reduced strain capacity--both elastic strain and creep strain--is not a desirable property in concrete (Mather 1993).

# 4 Shrinkage Test Methods

## Introduction

Our random survey of data sheets of cement-based materials produced in the United States shows that information about shrinkage is not even listed on some of them. Manufacturers tend to use different tests and standards to evaluate the performance of their products. Many tests are modified; some modifications were deficient or provided unrealistic results. Our survey of material data sheets revealed that USA manufacturers refer to eight shrinkage test methods:

*a.* ASTM C 596 (ASTM 1994m)

*b.* ASTM C 1107 (ASTM 1994x)

*c.* ASTM C 490 (ASTM 1994i)

*d.* ASTM C 157 (ASTM 1994e)

*e.* ASTM C 157 (Dry)

*f.* ASTM C 157 (with different modifications)

*g.* Ring test method (with different modifications)

*h.* DIN 52450 (German Standard) (Germany 1985)

Current construction needs have led to the production and marketing of a number of materials which are claimed to be expansive, shrinkage compensating, and nonshrinking. They are often specified for applications requiring the complete and permanent filling of cavities, joints, and voids between rigid surfaces.

To specify the appropriate material, to verify the manufacturer's claims, and to evaluate the performance of these products is almost impossible due to the variety of testing methods used to measure the shrinkage of the materials. In addition to the absence of a reliable industry wide testing method, manufacturers who are using the same standard method, for instance the widely used ASTM C 157, are modifying the method arbitrarily. The arbitrary application of any of these test methods has resulted in controversy and confusion in the selection and specification of materials.

Variations of the test methods including size of the specimen, restraint conditions, curing, time of initial readings, temperature and relative humidity limitations, and test duration further complicate the interpretation of comparative test results and properties indicated on manufacturers' data sheets.

The primary objective of this portion of the research study was to select or to develop an industry-wide reliable test method for drying shrinkage of cement-based materials. In the course of this study, three types of principal points were considered by the investigators:

*a.* How can a laboratory model test be devised which will be representative of the prototype structure when given a structure in a specified environment?

*b.* How can the results of a standard test on relatively simple specimens exposed to a controlled environment be interpreted to provide indications for a variety of structures in different environments? The important factors which will influence the relationship between test results and prototype performance fall into four primary categories: restraint conditions, environment, loading, and geometry.

*c.* How does one accurately define a standard environment or a number of standard environments for laboratory testing, when in the field practice no two environments are even relatively the same?

A general review of various test methods for evaluation of shrinkage, which is given next, will illustrate these points of view.

## Standard Test Methods Used in the United States

The following are a description and review of some of the standard test methods used for measuring various types of shrinkage.

*a.* ASTM C 1090, "Standard test method for measuring changes in height of cylindrical specimens from hydraulic cement grout" (ASTM 1994w). Specimens are fabricated in 3- by 6-in. cylindrical steel molds. The top surface of the specimen is finished and covered with a glass plate for 4 hr after the final set. Before removal of the plate, an initial reading is made from a reference plate to the glass surface with a depth micrometer. Specimens are autogeneously cured to prevent moisture loss, and volume change readings are taken at 3, 14, and 28 days after molding. This method measures only autogenous shrinkage.

*b.* ASTM C 827, "Standard test method for change in height at early age of clyndrical specimens from cementitous mixtures" (Light Projection Method) (ASTM 1994q). Grout is placed in 2- by 4-in. cylindrical steel molds and screeded flush. An indicator ball is placed in the center of each specimen and tapped lightly so that approximately one-half of its diameter penetrates the grout. The specimens are then placed in the projection apparatus and positioned so the images of the balls in the indicating charts are in sharp focus. Readings are taken at periodic intervals until the initial set and as often as desired thereafter. After the initial reading, a light coating of medium weight oil is applied to the grout surfaces to simulate autogenous curing.

*c.* ASTM C 157, "Standard test method for length change of hardened-hydraulic cement mortar and concrete" (ASTM 1994e). Mortar bars are fabricated in molds conforming to ASTM C 490 (ASTM 1994i) and having gage lengths of 10 $\pm$ 0.10 in. Length change is measured using a comparator and is expressed as a percent of the initial gage length after stripping. All mortar bars are left in the molds for the first 24 hr after fabrication. After stripping, the bars are measured and placed in lime water for an additional 28 days. Bars are then placed in an environmental room for cure at approximately 70 °F and 50 percent relative humidity for the specified duration of the test.

*d.* ASTM C 806, "Standard test method for restrained expansion of expansive cement mortar" (ASTM 1994p). Grout prisms are molded 2 in.$^2$ by 10 in. long between end plates 3/8 in. thick. The end plates are connected by a 1/4-in.-diam threaded mill steel rod. Stainless steel cap nuts, which fit into the length comparator, are attached to the ends of the threaded rod extending through the 3/8-in. plates. Molds are stripped from the bars 6 $\pm$ 1/4 hr after the addition of water and the bars are measured for initial length before placing in lime water for curing. Specimens are removed from the lime water for additional readings at 7 and 28 days.

Comparison of grout expansions (contractions) tested by different methods and coefficient of variations is shown in Tables 18 and 19. (Tables are based on results of the study (Best and Lane 1981)).

The following are general observations based on the analysis of the described test methods and test results (Best and Lane 1981):

*a.* ASTM C 1090, "Standard test method for measuring changes in height of cylindrical specimens from hydraulic cement grout" (ASTM 1994w). This method does not allow the determination of plastic volume change which may be desirable. Within-test variations with this method were less than with any of the others, but the variation in results from one test to another are considered rather large for use as a precision test method. The major disadvantage of the micrometer bridge method is its inability to measure plastic volume changes.

*b.* ASTM C 827, "Standard test method for changes in height at early ages of cylindrical mixtures" (Light Projection Method) (ASTM 1994q). The light projection method has an advantage of measuring both plastic and hardened phases volume changes. The disadvantages include the unrealistic curing effects of the oil coating, the relatively large within-test variations, and the lack of precision in establishing a zero base reading. The latter two may be related to the difficulty in establishing equilibrium of the indicator ball, since its final position may vary depending on the specific gravities of the grout and the ball, particularly for fluid mixes.

*c.* ASTM C 157 "Standard test method for length change of hardened-hydraulic cement mortar and concrete" (ASTM 1994e). This method is not recommended for qualifying premixed grouts, since the initial reading neglects the volume change during the first 24 hr. The applicability of this method is limited since specimens are completely unrestrained and the ratio of longitudinal to lateral dimensions is far greater than normally encountered in most grout installations.

*d.* ASTM C 806 "Standard test method for restrained expansion of expansive cement mortar" (ASTM 1994p). The restrained mortar bar method shares many of the disadvantages of ASTM C 157 (ASTM 1994e), although specimens are restrained in the direction of volume measurement and zero base reference lengths are read after approximately 6 hr as opposed to 24 hr. Plastic volume change is not accounted for by this method.

**Table 18**
**Comparison of Grout Volume Changes Tested by Different Methods (Best and Lane 1981)**

| | 2 days | | | | 7 days | | | | 28 days | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Grout Type | ASTM C 827 | ASTM C 621 | ASTM C 157 | ASTM C 806 | ASTM C 827 | ASTM C 621 | ASTM C 157 | ASTM C 806 | ASTM C 827 | ASTM C 621 | ASTM C 157 | ASTM C 806 |
| A (Nonmetallic) | -0.16 | 0.06 | -0.01 | 0.01 | -0.06 | 0.06 | 0.11 | -0.05 | - | 0.06 | 0.19 | -0.11 |
| B (Metal Oxidizing) | -0.67 | -0.4 | 0.03 | 0.01 | -0.67 | -0.4 | 0.16 | -0.06 | - | -0.38 | -0.2 | -0.06 |
| C (Sand-cement) Control Mix | -0.88 | -1.3 | -0.03 | -0.03 | -0.88 | 0 | -0.08 | -0.04 | - | -1.38 | -0.14 | -0.09 |
| D (Gas-producing) | 1.05 | -0.2 | 0.01 | 0 | 1.05 | -0.2 | -0.06 | -0.03 | - | -0.16 | -0.14 | -0.08 |
| E (Expansive cement) | -0.35 | -0.3 | 0.01 | 0 | -0.32 | -0.3 | -0.06 | -0.02 | - | -0.31 | -0.17 | -0.08 |

Notes: Data are expressed as a percentage of initial length.
ASTM standards information listed in References section following main text.

| **Table 19**<br>**Coefficient of Variation for Test Methods (Best and Lane 1981)** | | | | | |
|---|---|---|---|---|---|
| | | **Coefficient of Variation of 7-day Results, %** | | | |
| **Grout Designation** | **Grout Type** | **ASTM C 827** | **ASTM C 621** | **ASTM C 157** | **ASTM C 806** |
| A | Nonmetallic | 131.3 | 100.0 | 18.2 | 16.0 |
| B | Metal oxidizing | 13.4 | 10.5 | 33.3 | 16.7 |
| C | Sand-cement | 14.8 | 16.8 | 50.0 | 75.0 |
| D | Gas producing | 17.0 | 25.0 | 33.3 | 33.3 |
| E | Expansive cement | 53.1 | 80.6 | 3.3 | 5.0 |
| Note: ASTM standards information listed on References section following main text. | | | | | |

## ASTM C 928 (ASTM 1994u)

This is the standard specification for packaged, dry, rapid-hardening cementitious materials for concrete repairs. This specification is intended only for rapid hardening materials. The specified test method for length change presents the following modifications to the ASTM C 157 (ASTM 1994e):

*a.* Specimens are removed from the molds at an age of 2-1/2 to 2-3/4 hr after the addition of a mixing liquid to the dry cementitious mixture during the mixing operation.

*b.* The initial observation of length is made 3 to 3-1/4 hr after the addition of mixing liquid during the mixing operation.

*c.* The observation of length is done at age 28 days ± 20 hr. The average percent increase in length is determined when stored in water and the average decrease in length when stored in air.

The standard calls for shrinkage not to exceed 0.15 percent. Shrinkage of 0.15 percent is the very high range, being three times the shrinkage of normal concrete. It is our opinion that the repairs with such materials will be highly susceptible to excessive drying shrinkage stress, cracking, delamination, and failure.

## Free Shrinkage Test Methods

### Alberta Transportation and Utilities (ATU) Specifications B-391 (ATU 1992)

ATU Specification B-391, "Specification for bridge cement patching material" (ATU 1992) includes modifications to ASTM C 157 test method. This test method had the following limitations, according to ATU:

*a.* It measures no length change which occurs during the initial 24 hr.

*b.* The specimens are cured in saturated lime water for a period of 28 days before exposure to a 50 $\pm$ 4 percent relative humidity environment, after which length change is measured.

To provide and evaluate the materials that more closely represent field conditions, the following changes to ASTM C 157 have been incorporated:

*a.* Length changes are referenced to the gage length of the prism at the time of casting. Length change determinations are required at the ages of 1, 3, 7, 28, 60, and 120 days.

*b.* One set of specimens is required to be prepared using the manufacturer's recommendations for curing during the initial 24 hr to represent favorable curing conditions.

A second set of specimens is required to be cast and cured at 50-percent relative humidity for the initial 24 hr after casting, and at a relative humidity of 50 percent for the remainder of the test period. Comparison of the results of the tests for the two different curing regimes thus permits an evaluation of the likely effect on length change due to placement of repair materials at different locations at different times of the year. The mold used in the ATU testing program is shown in Figure 17.

The following test procedures were used in the evaluation program.

*a.* Length change testing was conducted using the test procedures detailed in ASTM C 157 (ASTM 1994e) and ASTM C 490 (ASTM 1994i) for 1.0- by 1.0- by 11.0-in. (25- by 25- by 285-mm) specimens. Prior to casting, the inside of the molds were sheathed with a thin polyethylene sheet to facilitate the removal of the specimens during demolding. Each end of the mold contained a gage stud which was held in place by a gage spacer, which in turn was connected by screws to the end plate. By releasing the screws connecting the end plate to the spacer and the base plate, the end plate, spacer and gage studs all become free to move horizontally with changes in the length of the mortar specimen. The gage length was adjusted using the screw action of the gage studs until the gage length conformed to the length of the reference bar. The

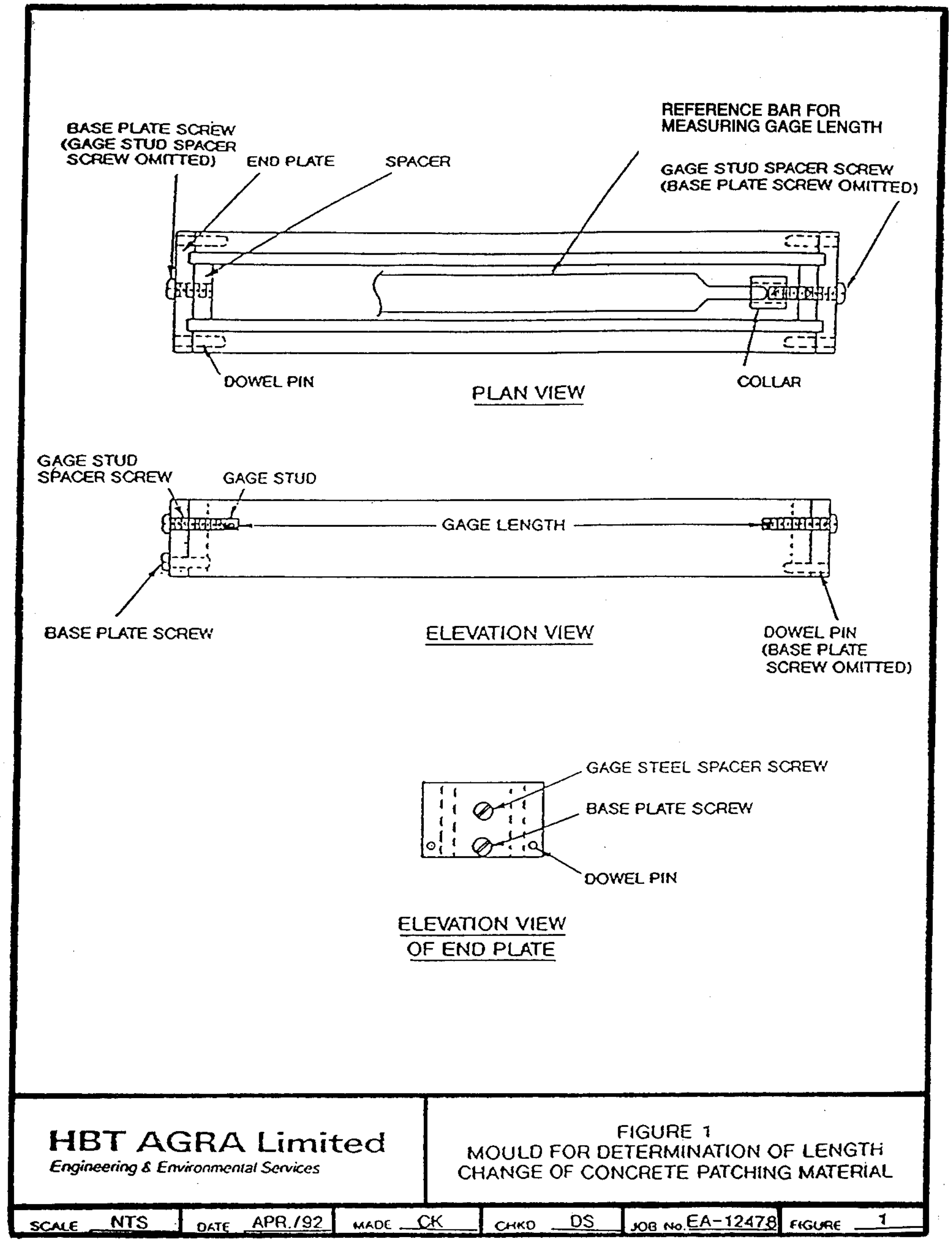



Figure 17. Mold for determination of length change of patching material (ATU 1992)

reference bar was similar to the one used in the comparator for determining the length changes of the hardened mortar specimens, with the exception that the length was somewhat shorter to fit between the ends of the gage studs. The length of the gage studs was measured with a suitable micrometer to allow the calculation of the total length between the ends of the studs. The screws holding the gage stud were subsequently tightened to prevent movement of the gage studs until the mortar surrounding the studs had set.

*b.* The manufacturers' recommendations were used for proportioning and mixing the materials. Two sets of the prisms were cast using the procedures described in ASTM C 157. The approximate time of the final set of the batch was determined using a Vicat apparatus.

*c.* At the time of the final set, the screws holding the end plate, gage stud spacer, and gage studs were released, thereby allowing the gage studs to move with length change of the mortar. During the first 24 hr of curing, one set of specimens for each product tested was cured in a moisture chamber maintained at 50 percent $\pm$ 4 percent relative humidity (RH) and 23 $\pm$ 3 °C (73 $\pm$ 3 °F). The second set was cured during the first 24 hr according to the manufacturer's instructions, or exposed to laboratory ambient conditions if curing instructions were not provided. All of the specimens were demolded after 24 hr of initial curing and stored in a moisture chamber at 50 percent $\pm$ 4 percent RH and a temperature of 2 $\pm$ 3 °C (73 $\pm$ 3 °F) for the remainder of the test period.

*d.* After demolding, the specimen length was measured and compared to the gage length at the time of casting. Comparator readings were subsequently taken, in accordance with the procedures in ASTM C 157, to determine the length change at ages 3, 7, 28, 60, and 120 days.

## Structural Preservation Systems, Inc.

This test method (Figures 18, 19, 20) incorporates a 10-ft. (3-m)-long, 4-in.-diam split PVC form with stainless steel angles at each end. Shrinkage is measured at the edge opposite the location of the restraining anchor.

## Staynes Method (Staynes and Willway 1987)

Measurement of early age shrinkage of resin-based materials in the United Kingdom is carried out by the Staynes method, Figure 21 (Staynes and Willway 1987). This method has since been modified for cementitious mortars (Emberson and Mays 1990). Although still in its infancy, this type of measurement could one day provide a much clearer picture of shrinkage almost from the time of initial mixing. As yet, a criteria for the acceptable

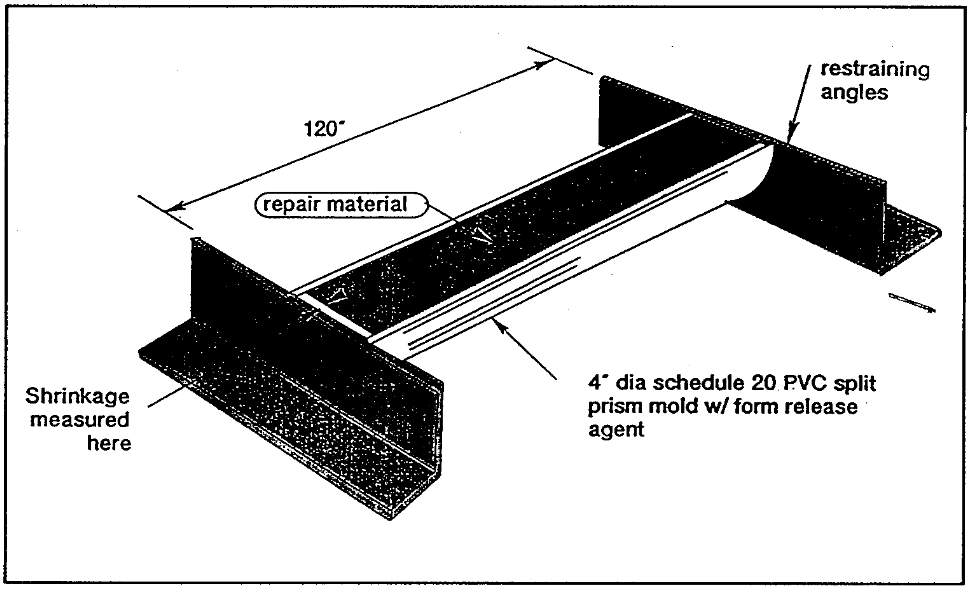



Figure 18. General view of the SPS test assembly

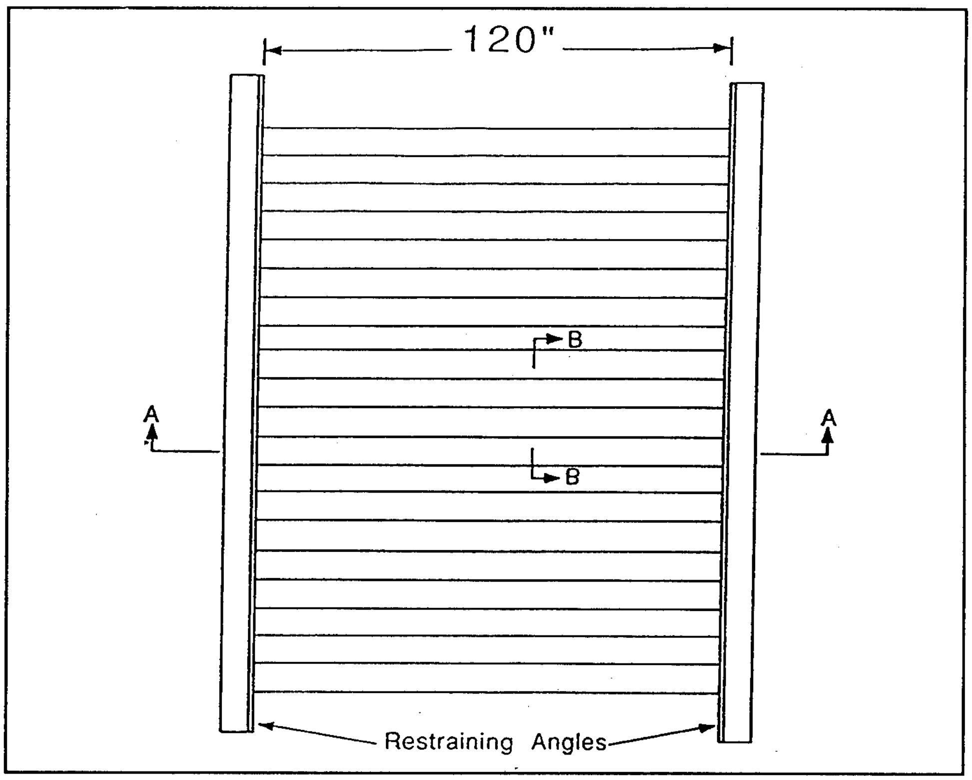



Figure 19. Plan view of test specimens. Patching material shrinkage test

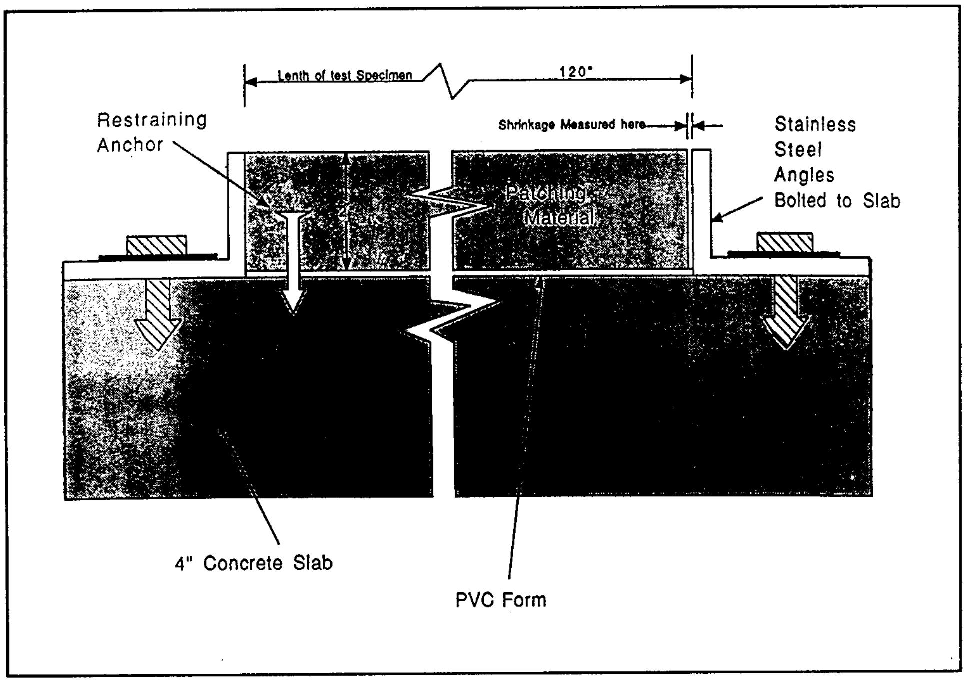



Figure 20. Section AA through specimen. Patching material shrinkage test

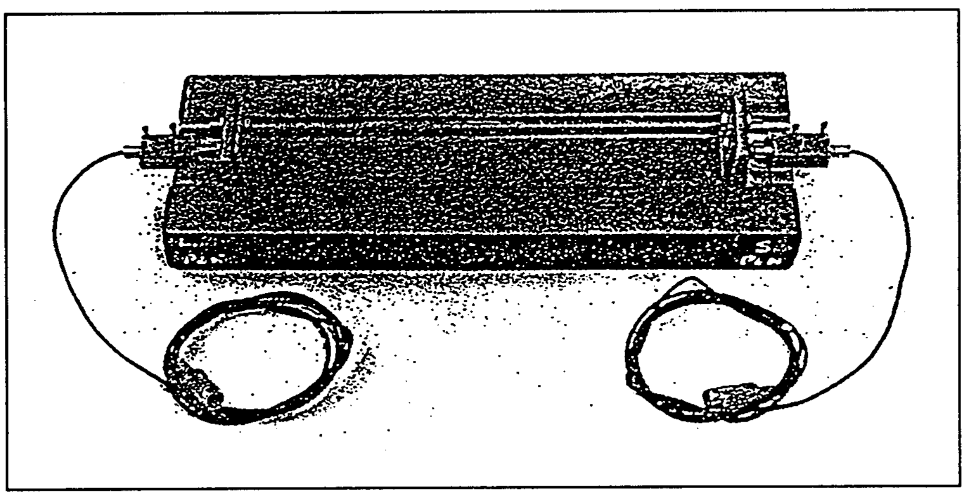


Figure 21. Test equipment developed by Staynes and Willway (1987)

performance of a repair mortar in this test have not been set, and the method is still being studied.

## Restrained Shrinkage Tests

Techniques to examine restrained shrinkage generally rely on the cracking of the material within a given time period. There are three basic types of restrained shrinkage tests that have been used for cement-based materials - linear, plate, and ring.

In the linear restrained shrinkage test, restraint of the movements of the specimens is provided either internally by axially embedded bars or tubes or externally by a large steel mold or frame (Blakey 1958, Blakey and Lewis 1959, Glanville 1930, Thomas 1936, Carlson 1938 and 1940, Fujimatsu 1958).

In the plate type, the restraint is provided only at the bottom of the prismatic specimens. The plate-type test is convenient when the aggregate size is small {less than about 5 mm (0.2 in.)} and has generally been used in conjunction with neat cement paste or mortar specimens.

In the ring-type test, the cement-based material is cast around a ring of steel, and then it dries on three sides while the metallic ring provides the necessary restraint.

The following is an illustration of some of the restrained shrinkage tests.

### Ring method

*a.* Civil Engineering Department, University of Sheffield, England (Swamy, Bandyopadhyay, and Stavrides 1979)

This ring test incorporates several new features, although it is based on similar tests by earlier investigators who also cast concrete around a ring of steel. The central steel ring in this method not only provides restraint to shrinkage during drying, it also serves as a sensitive dynamometer for measuring the induced stresses. The size of the ring was determined by three main considerations: (1) the need to avoid large differential shrinkage within the tested specimen, (2) the need to accommodate aggregate sizes up to a maximum of 25 mm (1 in.) and short fibers up to 50 mm (2 in.) long; and (3) the need for the ring to have the same volume/surface ratio as that of the control test specimens, which were 102 by 102 by 508 mm (4 by 4 by 20 in.). These considerations led to the adoption of the test specimen shown in Figure 22, which had a cross-sectional area of 76 by 102 mm (3 by 4 in.) and an inner diameter of 508 mm (20 in.).

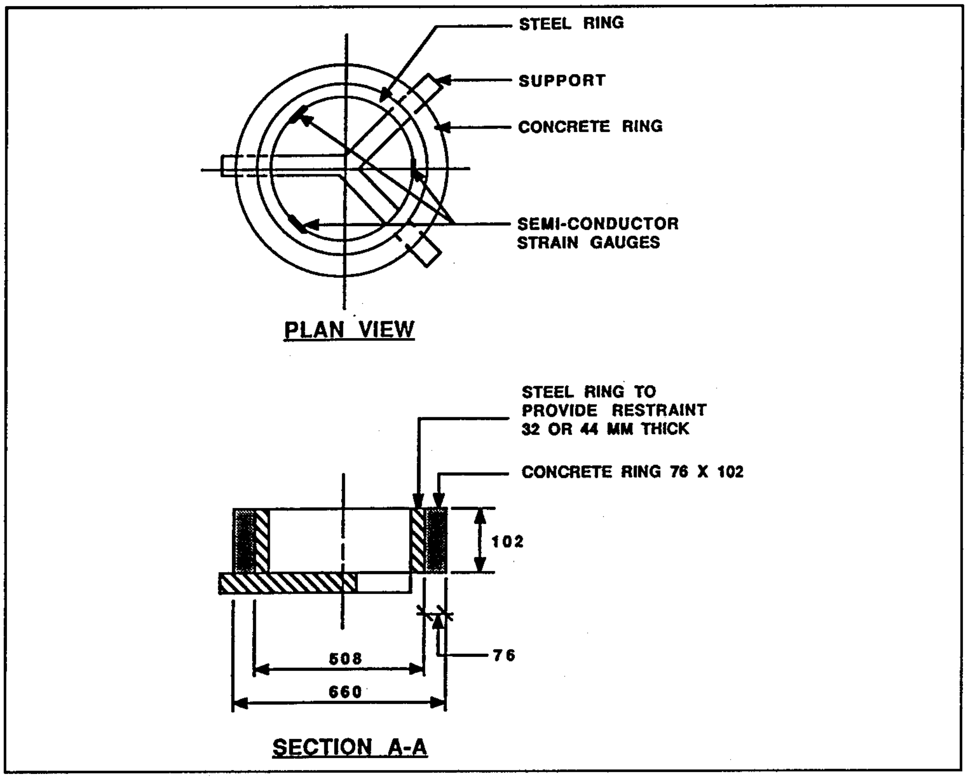



Figure 22. Details of the mold for the Restrained Shrinkage Test (Swamy, Bandyopadhyay, and Stavrides 1979)

The dimensions of the steel ring were determined by two considerations. First, since the order of magnitude of the restrained shrinkage stresses is generally small, it was important that the restraining device be neither too rigid to measure the induced stresses nor too flexible to provide adequate restraint to the concrete ring. The steel ring was therefore designed in two thicknesses, 44 mm (1.74 in.) and 32 mm (1.26 in.) to limit the movement of the restrained concrete to about 10 percent of the free shrinkage or about 50 to 70 $\mu$m/m. The size of the steel ring used in any particular test was based on the free shrinkage of the material being tested. In general, the 44-mm (1.74-in.)-thick steel rings were used for high early strength cements such as the aluminous and ultrafine cements and the 32-mm (1.26-in.)-thick steel rings were used for ordinary Type I portland cements.

Each ring frame was supported in three steel arms so that the concrete ring could dry from the lower face as well. To facilitate early dismantling of the ring, the bottom shuttering for the rings was made in three segments of 25-mm-thick plywood joined together over the radial steel supports. The outer steel ring was made in two halves by rolling 5-mm (0.2-in.)-thick mild steel plates, which were then firmly clamped to the bottom wooden base and the radial arms. The inner steel ring was also firmly clamped to the base of the mold.

Tests were carried out under controlled temperature (16 ± 1°) and humidity (50 ± 2 percent) conditions. The exposed surfaces of the concrete allow cracking caused by restrained shrinkage to be quantified.

*b.* National Science Foundation (NSF) Science and Technology Center for Advanced Cement-based Materials (Shah, Karaguler, and Sarigaphuti 1992)

The dimensions of the specimens used in this study are given in Figure 23. Using these dimensions, it can be shown that if the concrete ring is subjected to an internal pressure caused by the restraint provided by the steel ring, then the difference between the values of the tensile hoop stresses on the outer and inner surfaces is only 10 percent. Also, the maximum value of the radial stresses is only 20 percent of the maximum hoop stress. Thus, it can be assumed

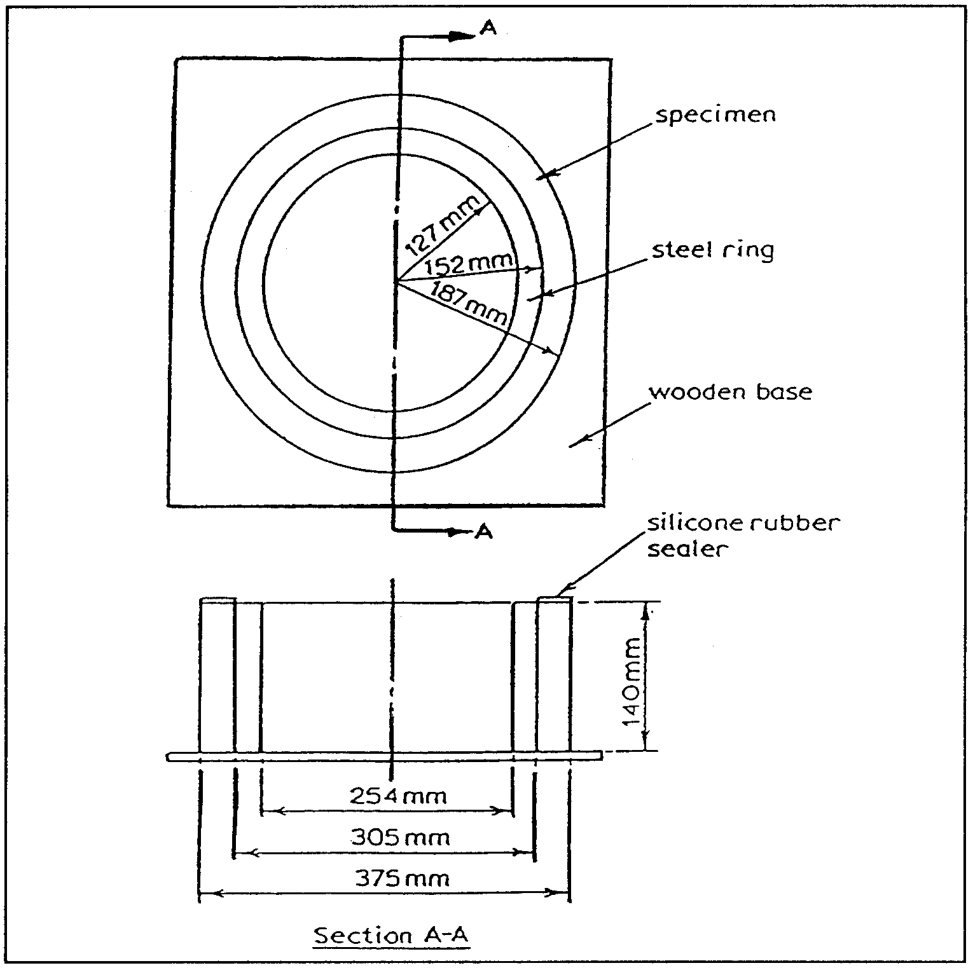



Figure 23. Dimensions of the test specimen (Shah, Karaguler, and Sarigaphuti 1992)

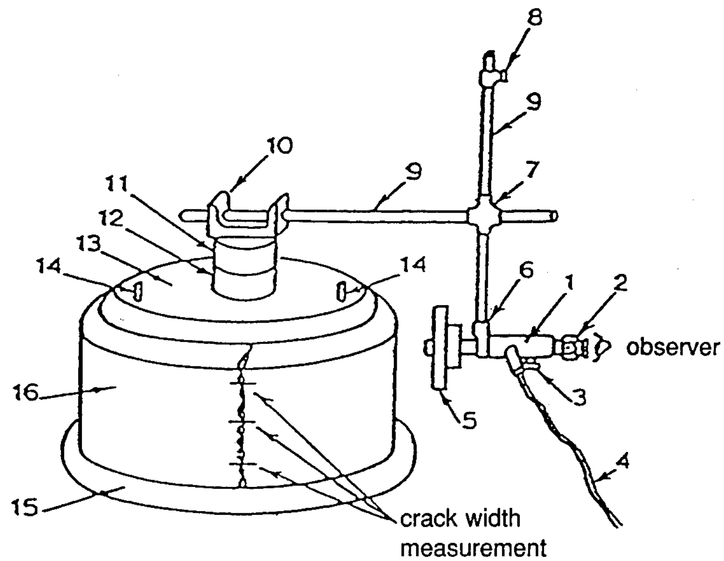
that the concrete annulus is subjected to essentially uniform, uniaxial, tensile stress when it is internally restrained by the steel ring, provided the effects of nonuniform drying are negligible.

Drying was allowed from the outer circumferential surface. In addition, the width of the specimen {140 mm (5.5 in.)} was four times its thickness {35 mm (1.38 in.)}. As such, uniform shrinkage along the width of the specimen can be assumed.

The inner steel ring was obtained by cutting a round mechanical (steel) tube. A PVC tube was used as an outer mold. These two rings were placed concentrically on a wooden base so that the annular space between them could be filled with a concrete mixture. The outer mold was stripped 4 hr after casting. Then, the top surface of the concrete ring was sealed using silicon rubber so that the drying would be allowed only from the outer circumferential surface. After that, the specimen was exposed to drying in the humidity room at 20 °C (68 °F), 40 percent RH.

Cracking in restrained concrete was investigated between 4 hr and 42 days. Both the restrained shrinkage ring specimen and the prism for measuring free shrinkage were cured for 4 hr at 20 °C (68 °F), 100 percent RH, and then after demolding exposed to drying in the humidity room at 20 °C (68 °F), 100 percent RH. The curing time of 4 hr was chosen to measure shrinkage as early as possible. At this time, concrete was just strong enough to remove the mold.

To measure crack width, a special microscope setup was designed (Figure 24). The microscope was fixed to an adjustable, scaled locator connected to the round steel plate installed on the top of the specimen. The ballbearing on the top of the plate enabled the microscope to move around the specimen, whereas the locator was connected to a horizontal bar, permitting up-and-down movement so that the whole circumferential surface of the specimen could be observed with the microscope. The crack width reported was an average of three measurements; one at the center of the ring and the other two at the top and bottom half of the ring (Figure 24). The surface of the specimens was examined for new cracks and the measurements of the widths of existing cracks were performed every 24 hr during the first 7 days after cracking, and then every 48 hr.




1 microscope
2 filar eyepiece
3 focusing knob
4 internal illumination
5 fluorescent lamp
6, 7, 8, 9, 10, 11 microscope mounting hardware
12 preloaded bearing pivot
13 mounting plate
14 dowel pins
15 wooden base
16 specimen

Figure 24. Test setup to measure crack width with microscope

*c.* Method Used by Technical Research Centre of Finland (VIT)[1]

Instructions for Testing Restrained Shrinkage

(1) Premixed concretes, cement- and polymer-based batch grouts, jointing mortars, polymer-based jointing mastics and screeds.

- Principle

The widths of cracks developing over specific periods of time in a layer of the material cast around a metal tube are to be measured.

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1 Personal Communication, 27 April 1993, George Tessier, Manager R & D, Thoro System Products, Bristol, PA.

The method is applicable when the maximum particle size is not greater then 8 mm.

- Test specimens, their fabrication and curing

The formation of cracks during restrained shrinkage is to be studied using hoops (external diameter = 155 mm) cast around a steel tube (diameter = 115 mm, h = 50 mm). Two hoops shall be fabricated.

The hoops are to be kept in their molds and covered with plastic for the first 24 hr after they are cast. Having been removed from their molds, the hoops are to be kept until they are 7 days old in water at a temperature of 20 ± 2 °C or an environment with a relative humidity of 70 ± 5 percent and a temperature of 20 ± 2 °C, depending on which is more favorable for the hardening of the material. After this, they are transferred to an environment with a relative humidity of 70 ± 5 percent and a temperature of 20 ± 2 °C. Then, when 28 days old, they are transferred to an environment with a relative humidity of 40 ± 5 percent and a temperature of 20 ± 2 °C, where they are stored until 56 days old.

- Test procedure

During the test, the cracking of the hoops is examined so that the time of the crack formation can be determined with an accuracy of ± 3 days and also so that it is known in which storage conditions the cracks originated. When the hoops are 56 days old, the widths of the cracks are measured with an accuracy of 0.1 mm at the outer edge of the hoops.

- Test results

The time of crack formation with an accuracy of ± 3 days and the number and widths of the cracks with an accuracy of 0.1 mm at an age of 56 days are to be reported as the test results.

(2) Premixed dry concretes

- Principle

The widths of cracks developing over specific periods of time in a layer of the material cast around a metal tube are to be measured. The method is applicable when the maximum particle size is greater than 8 mm.

- Test specimens, their fabrication and curing.

  The formation of cracks during restrained shrinkage is to be studied using hoops (external diameter = 550 mm) cast around a steel tube (diameter = 300 mm, h = 150 mm). Two such hoops shall be fabricated.

  The hoops are to be kept in their molds and covered with plastic for the first 24 hr after they are cast. Having been removed from their molds, the hoops are to be kept until they are 7 days old in either water at a temperature of 20 ± 2 °C or an environment with a relative humidity of 70 ± 5 percent and a temperature of 20 ± 2 °C, depending on which is more favorable for the hardening of the material. After this, they are transferred to an environment with a relative humidity of 70 ± 5 percent and a temperature of 20 ± 2 °C. Then, when 28 days old, they are transferred to an environment with a relative humidity of 40 ± 5 percent and a temperature of 20 ± 2 °C, where they are stored until 56 days old.

- Test procedure

  During the test, the cracking of the hoops is examined so that the time of the crack formation can be determined with an accuracy of ± 3 days, and so that it is known in which storage conditions the cracks originated. When the hoops are 56 days old, the widths of the cracks are measured with an accuracy of 0.1 mm at the outer edge of the hoops.

- Test results

  The time of crack formation with an accuracy of ± 3 days and the number and widths of the cracks with an accuracy of 0.1 mm at an age of 56 days are to be reported as the test results.

*d.* Fosroc Corporation (Rizzo and Sobelman 1989)

Shrinkage ring test method is used. The dimensions of the specimens used are as follows: inner restraining ring - 4-1/2-in. O.D., outer ring-7-in. I.D. to give a specimen 2-1/2 in. wide and 2 in. deep (Figure 25).

*e.* Fibrous Concrete Early Age Shrinkage Testing (Padron and Zolla 1990).

This test method was developed at the Civil Engineering Department, University of Miami, Coral Gables, FL, and, in our view, it presents a modification of the ring method.

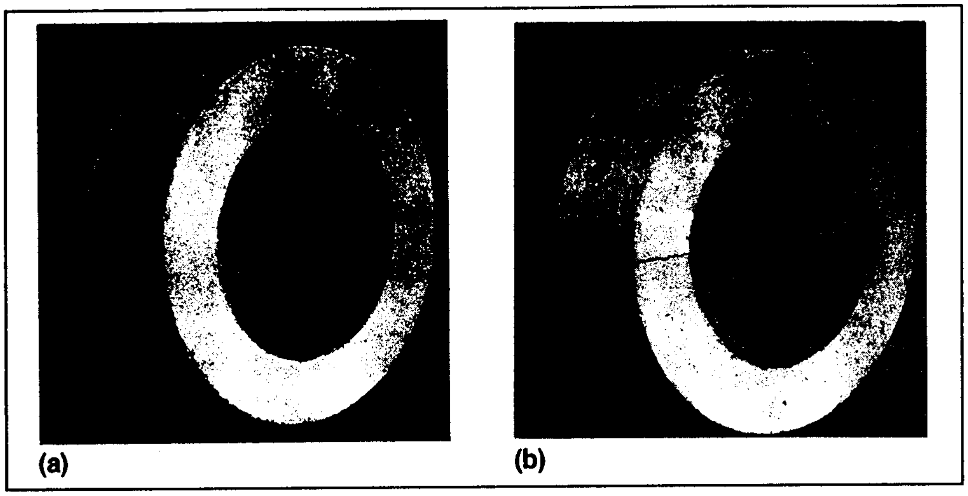


Figure 25. Restrained shrinkage tests were carried out on two cement-based repair materials exposed to 70 °F and 65 percent relative humidity: (a) a unique repair after 1 year; (b) another material after 7 days (Rizzo and Sobelman 1989)

Approximately 1-ft. (0.09-m) square plate specimens were chosen from either 1/2- or 1-in. (12- or 25-mm) thicknesses, with the thicker plates being used for preparation of large aggregate samples. Restraint against shrinkage was provided by a steel ring placed at the center of the sample form prior to casting the samples. These steel rings were cut from standard steel pipe of 4.5-in. (114.3-mm) diam for the 1/2-in. (12.6-mm) plate specimens and 5.5-in. (139.7-mm) diam for the 1-in. (25.4-mm) specimens. The rings were then machined to the exact thickness of the forms and specimens to be cast.

Sample specimens were cast in transparent plastic molds in a horizontal position. The molds were impermeable, nonabsorbent, and reusable. A thin coat of silicone wax was applied to the plastic molds prior to casting the specimens. This was done to facilitate the removal of specimens from the molds after testing and to help prevent the mold bottoms or sides from acting as sources of restraint for the specimens undergoing volume change during the test program. A wind chamber capable of providing a steady flow in a designated test section was provided.

The test section was 27 in. wide and 5 ft high for a 135-in.$^2$ cross section. The effective length dimension in the direction of the wind stream was approximately 48 in. (1,220 mm), and the overall length of the chamber was 88 in. (2,235 mm). The chamber was constructed with three sides of transparent plastic sheet material placed on a table top. It was enclosed with gasket-sealing material and the edges taped to prevent air leakage.

Air flow was controlled by an 18-in. (458-mm) fan attached to a variac for speed control and so placed to exhaust the wind chamber, i.e., at the upstream side of the test section and samples. A tube capable of being moved to various locations in the roof of the chamber was used to determine flow velocities. A large section of the roof directly over the test section was designed to be removed to facilitate the placement of test samples within the test chamber.

The entire wind chamber was located in a room of approximately 972 $ft.^3$ (27.5 $m^3$) in which temperature and relative humidity could be controlled. Samples for shrinkage testing were placed in the wind chamber immediately after casting.

All mixing took place in a single-batch pan-type mixer with batches of sufficient size to cast three of the 1-$ft.^3$ (0.09-$m^3$) samples for shrinkage testing and three 3- by 6-in. (76- by 152-mm) cylinders for compression testing. Samples for shrinkage testing were placed in the wind chamber immediately after casting. Compression cylinders were permitted to set for 24 hr without moisture loss, then removed from molds and placed in the moisture room at 100 percent relative humidity and 82 °F (28 °C) for the completion of the 28-day curing cycle.

Mortar specimens were exposed to a wind stream velocity of 9 ft/s (2.7 m/s) {approximately 6 mph (9.7 km/hr)}. Concrete specimens (pea-gravel mix) were exposed to a 20-ft./s (6.1 m/s) wind stream velocity {approximately 14 mph (22.5 km/hr)}. Exposure in the chamber was continuous for 16 hr at a temperature of 88 °F (31 °C) and a relative humidity of 50 percent.

Initial cracks were observed after 1-3/4 to 2 hr for mortar specimens and from 1-1/2 to 2 hr for concrete specimens after drying had commenced. Most of the cracking was observable within the first 6 hr. Panels that had been stored in the dry room for 3 and 6 months after exposure in the wind chamber for 16 hr did not show measurable increases in crack area when reevaluated.

For the series of tests reported herein, specimens were placed in the wind chamber in sets of three, with two positioned across the width of the chamber perpendicular to the wind direction and the third centrally placed downstream from the direction of flow and the two upstream specimens. The objective of this placement was to prevent, as much as possible, dragging air (which had already crossed the upstream specimens and which would have picked up moisture) from crossing over the third (downstream) specimen.

Immediately after the 16-hr wind chamber exposure, the samples were polished by hand, using four successively finer grades of sandpaper, to improve the visibility of cracks. The sandpaper was stroked over the surface at right angles to each preceding direction of sanding with coarser paper grade. Grades 60 (coarse), 80 (medium), 150 (fine), and 220 (very fine) were used.

Measurements were taken as soon as possible after the polishing procedure was completed. Two measurements were targeted: a measure of the overall shrinkage of the panel and a measure of the total crack area on the exposed surface of the panel. In the former, the amount that the sample edges receded from the sides of the forms was used as a measure of shrinkage. This was determined as a total area of shrinkage cracking and was represented by the crack area between the exposed surface edge of the samples and the top surface of the form edges. Figure 26 shows the measured crack area. In this case, and in a similar manner of sample surface-crack area evaluation, crack areas

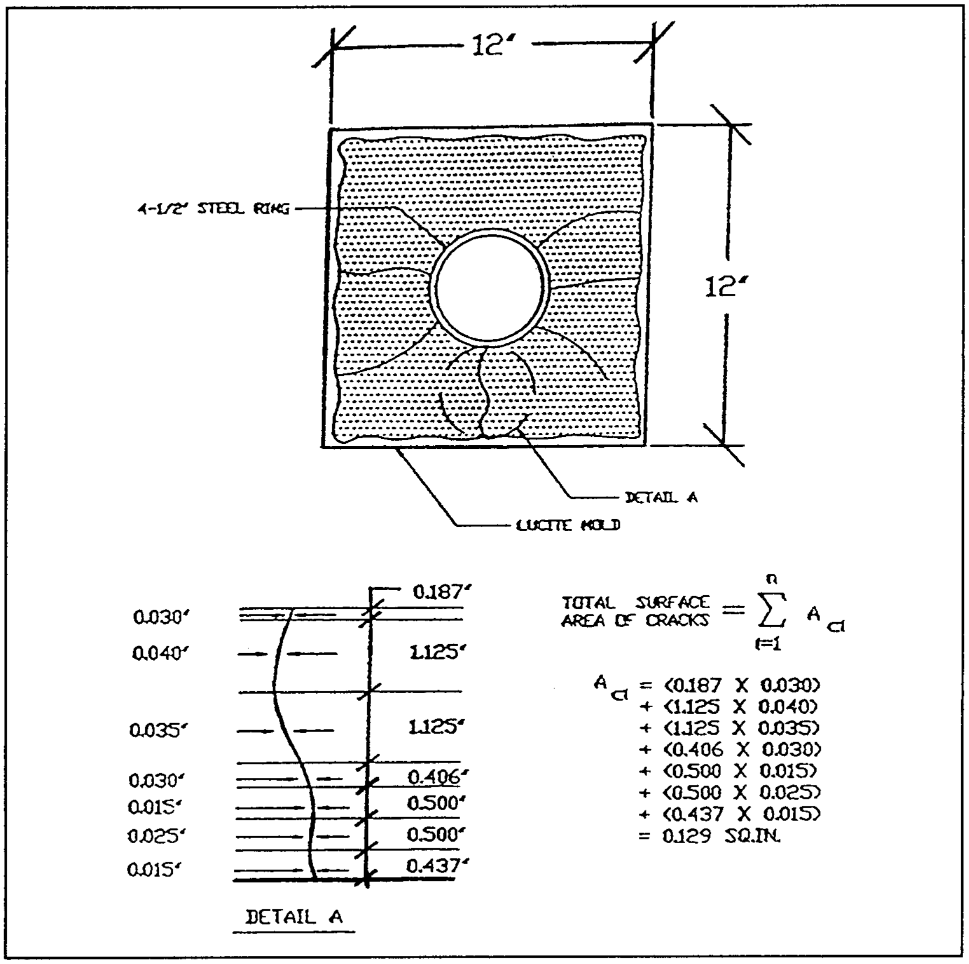


Figure 26. Typical specimen for measurement of overall shrinkage (1 in. = 25.4 mm) (Padron and Zollo 1990)

were determined by using a crack comparator and recording the total length of crack found for a given crack width. Figure 27 shows the distribution of crack widths available on the comparator used. Cracks were separated into four categories: large - 0.025 to 0.050 in. (0.64 to 1.27 mm), medium - 0.009 to 0.025 in. ( 0.22 to 0.64 mm), small - 0.005 to 0.009 in. (0.13 to 0.22 mm). The large, medium, and small

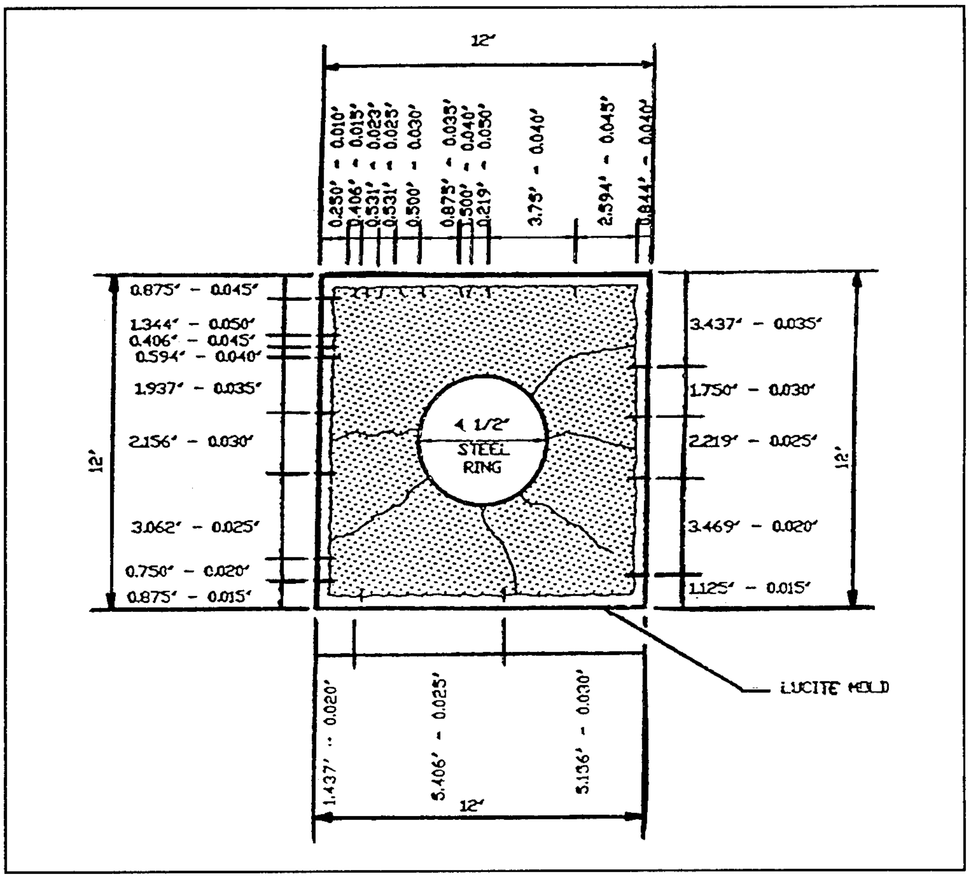


Figure 27. Typical specimen for measurement of shrinkage crack area (1 in. = 25.4) (Padron and Zollo 1990)

crack lengths were determined first. The specimen then was rescanned for hairline cracks.

### Linear restraint method

*a.* Federal Ministry for Transport, Germany (German Federal Ministry for Transport, Highway Construction Department 1990)

This method was developed by the Technical Academy, Aachen, Germany (Rheinisch-Westfalische Technische Hochschule Aachen) and adopted as the Technical Test Regulations (TP BE-PCC) for concrete substitution systems made of cement mortar/concrete with a plastic additive by the Highway Construction Department of the Federal Ministry for Transport.

After pouring mortar into the shrinkage channels (Figures 28 and 29), unless the manufacturer/applicant recommends otherwise, it is to be compacted by vibration. The compaction time, frequency, and

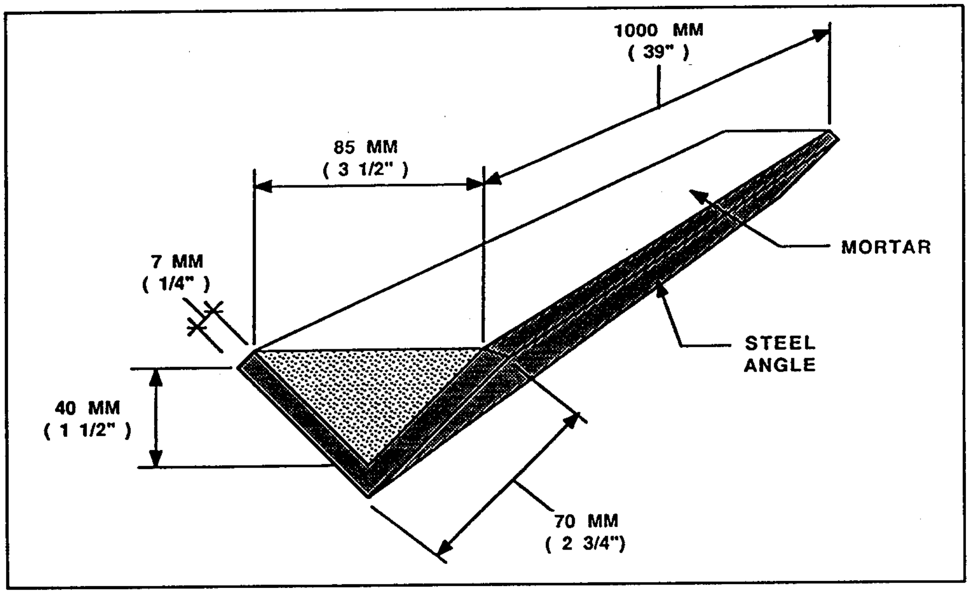


Figure 28. German Angle Method. Principal view of a filled angle (German Federal Ministry for Transport, Highway Construction Department 1990)

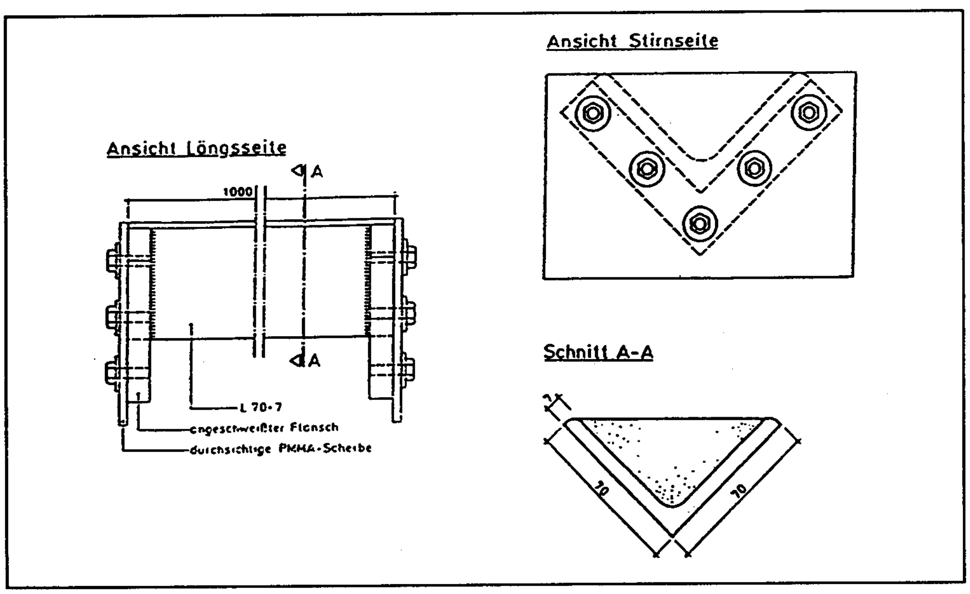


Figure 29. German Angle details. Abmessungen und Lage der Schwindrinne beim Einfullen des Mortels, MaBe in mm (German Federal Ministry for Transport, Highway Construction Department 1990)

amplitude are to be stated. As a rule, the mortar is to be leveled off and smoothed. The shrinkage channels are then to be kept uncovered in standard atmosphere. They are to be continually observed with a view to crack formation.

After 90 days, any cracks having occurred are to be measured precisely to within 0.02 mm (Figure 30). The number of cracks, the average and maximum crack width to within 0.02 mm, and, where applicable, the time of cracking, and large areas of detachment from the steel are all to be stated.

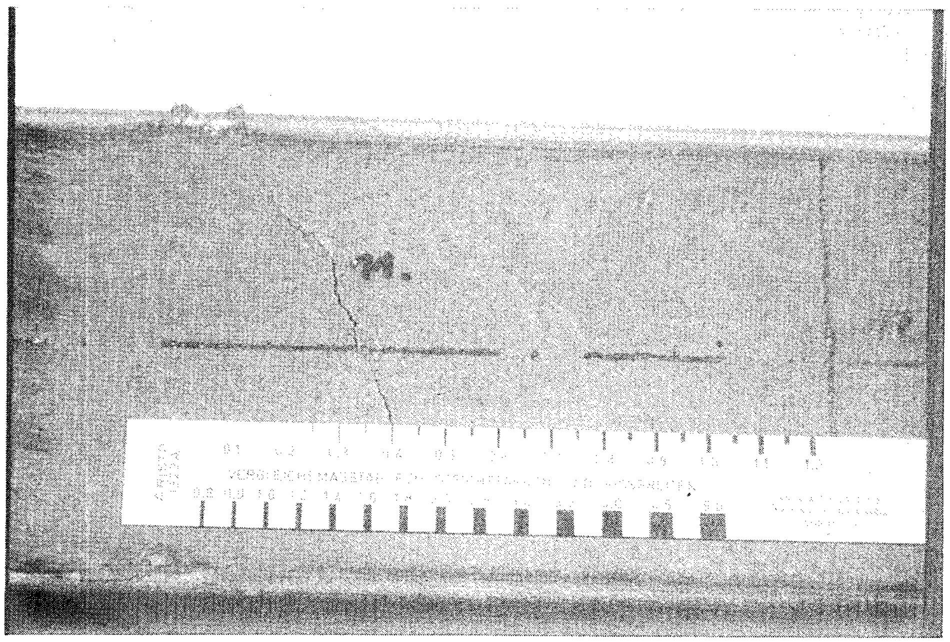

Figure 30. Measurement of crack width

Materials tested by this method with cracks wider than 0.1 mm are not accepted. Also, no bonding failure in large areas is allowed.

*b.* Thoro System Products[1]

This method is also based on visual observation and crack measurement of repair materials cured under ambient conditions. It incorporates several new features. For example, the geometry is more representative of concrete surface repair and the reinforcing steel is incorporated in the sample (Figures 31 and 32).

[1] Personal Communication, 27 April 1993, George Tessier, Manager R & D, Thoro System Products, Bristol, PA.

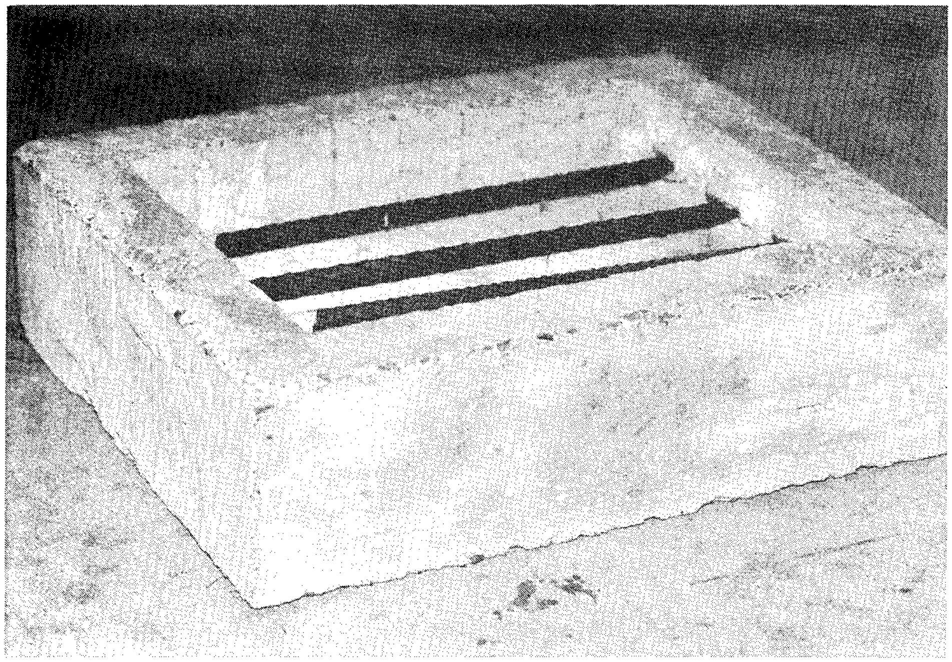

Figure 31. General view of Thoro Shrinkage Test slab

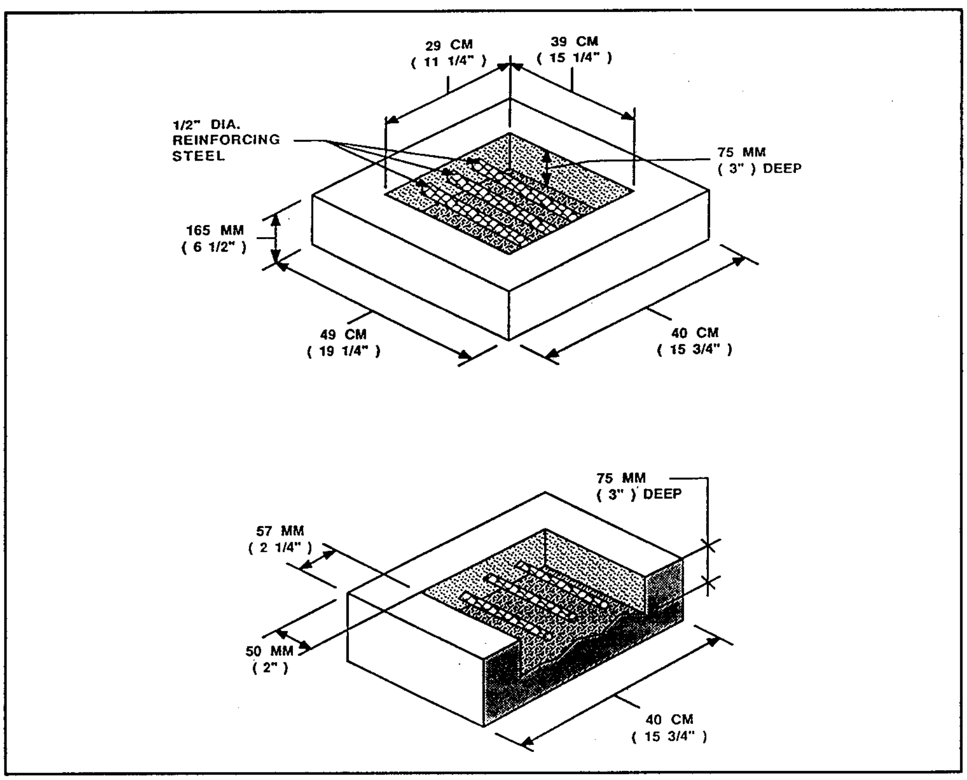


Figure 32. Thoro Shrinkage Test slab

*c.* Fosroc Corporation[1]

This method is based on visual observation and crack measurement of surface repair under ambient conditions (Figures 33 and 34).

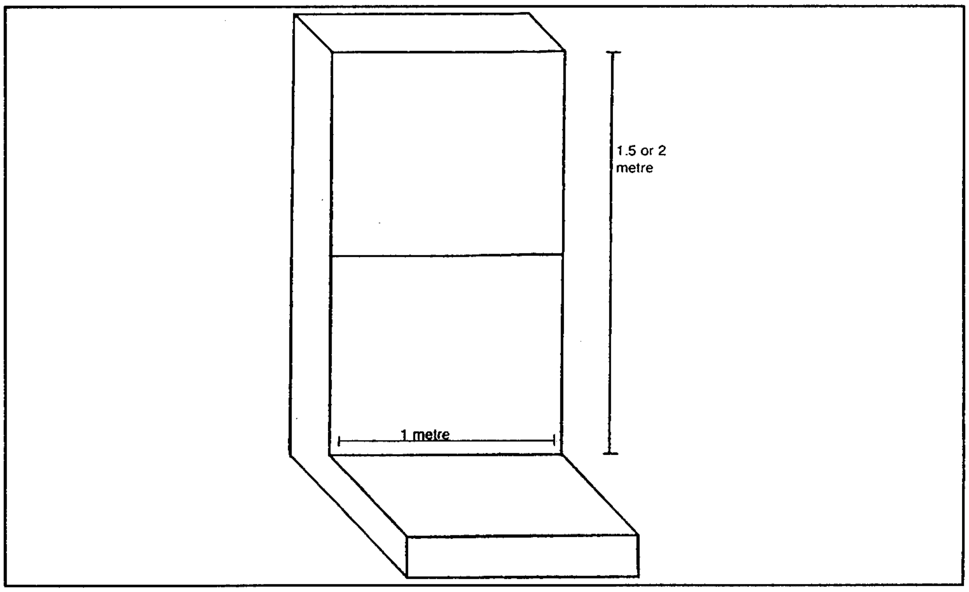


Figure 33. Fosroc freestanding concrete panel for testing repair systems

### Test for cracking potential due to drying shrinkage (Kraai 1985)

The test method proposed is not for measuring drying shrinkage, but rather for evaluating the cracking potential. For the test, 2- by 3-ft, 3/4-in.-thick specimens are used. The cracking potential is determined by comparing the cracking of the two test panels exposed simultaneously to a set of severe conditions designed to cause cracking. One panel is the control panel, the other is a similar panel except that a single material is altered to study its effect. The second panel could also be made identical in materials but then be subjected to different environmental conditions (temperature or drying conditions).

For the control panel, only those influences that are thought to maximize the amount of cracks on drying shrinkage cracking are chosen. Movement of free water is only in an upward direction. The bottom of the form is covered with polyethylene film to prevent absorption of water at the bottom. This film also prevents the bottom surface from restraining the volume change of the concrete. Additionally, bleed water moves up toward the top surface.

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[1] Personal Communication, 8 October 1993, M. H. Decter, International Project Manager, Fosroc International Ltd., Birmingham, England.

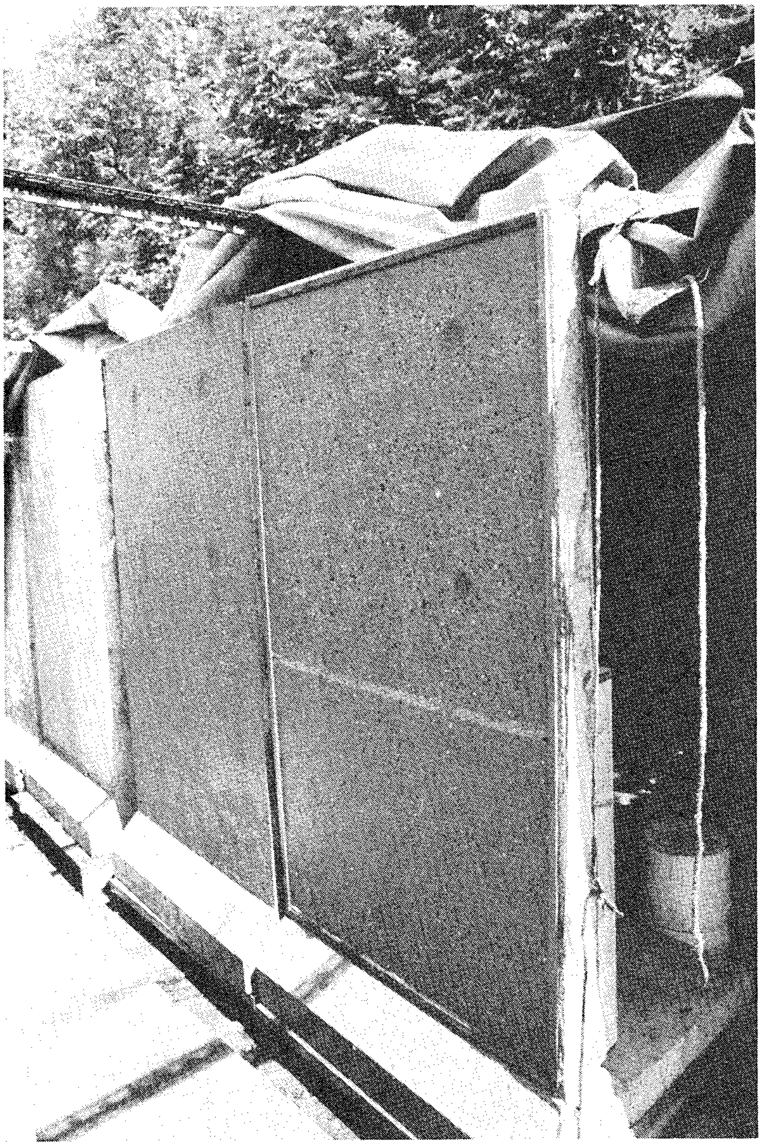

Figure 34. Panel application trial for repair material

A strip of 1/2- by 1-in. mesh hardware cloth is bent to an L-shape and installed just inside the perimeter of the form. This restrains movement caused by drying shrinkage which is the direct cause of cracking.

Immediately after casting, the two panels are exposed to a wind of 10 to 12 mph for 4 or 5 hr.

As soon as a concrete mix has been placed in the form, it is screeded and troweled with a 40-in. length of 1-in. angle iron that has been polished on one side to produce a smooth surface. In the current version of the test, both panels are set at the same time in front of fans (Figure 35) that produce the drying effect. To direct the wind onto the plastic concrete surfaces, the fans

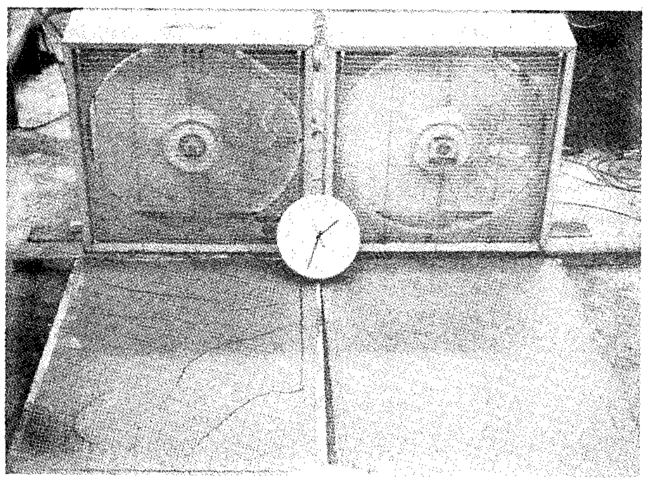

Figure 35. Test specimens after 2-1/2 hr of testing (Kraai 1985)

are tilted by means of a 3/4-in. piece of molding placed under their back edges. Ambient conditions including air speed need to be the same for the two panels being compared. Exposure to the induced wind should continue for 5 hr. Initial cracking begins to occur at the age of about 1 hr. Most of the cracking occurs within 4 hr.

Panel evaluation starts at 24 hr. At that time cracks are likely to be slightly wider and possibly more numerous than after 4 hr. Panels re-evaluated at 3 and 6 months have shown no significant further change in cracking patterns.

Crack lengths and average crack widths are measured and recorded for each panel. Crack widths fall into four categories, each of which is assigned a weighted value:

| Crack Width | Weighted Value |
|---|---|
| Large (about 3 mm) | 3 |
| Medium (about 2 mm) | 2 |
| Small (about 1 mm) | 1 |
| Hairline (about 0.5 mm) | 0.5 |

For each crack, the weighted value is multiplied by the crack length in millimeters to give the weighted average value. The sum of the weighted average values of all cracks in a panel is called the "total weighted average value" for the panel. This represents the cracking potential.

## Discussion

In the course of our study of state-of-the-art testing of shrinkage of cement-based materials to arrive at recommendations of what shrinkage test or tests should be used, some additional information on the subject was found. This information is discussed in the following paragraph for the purposes of this project.

Tremper and Spellman (1963) found that 10-in. (250-mm)-long specimens with a cross section similar to the specimens used in the ASTM C 157 (ASTM 1994e) length change test method were satisfactory for their extensive study of both cement mortars and concretes. Kraai (1982), however, argued that the ASTM C 157 method for measuring shrinkage of 4- × 4- × 10-in. (100- × 100- × 255-mm) concrete prisms is a laboratory test and that field data can range from 100 to 200 percent of the laboratory data.

Ytterberg (1987) describing his company's experience in using the ASTM C 878 (ASTM1994r) expansion test, agrees with Kraai that ASTM should adopt a field shrinkage test for concretes. The ASTM C 157 test requires a constant temperature as well as stripping the sample from the mold at an early age and placing it in lime water. Length measurements begin at 7 days when the samples are unnaturally saturated and expanded.

Bazant et al. (1987) reported a concrete shrinkage test program where they cast their samples in commercially available waxed paper cylindrical molds and kept them sealed in the molds until the instant of exposure to the drying environment at 7 days, thus eliminating the problems of sample stripping and moist curing found in the ASTM C 157 test. They found that long-time shrinkage can be predicted on the basis of 3- to 4-week shrinkage measurements of their special 83-mm-diam by 166-mm-long (3.27- by 6.53-in.) cylinders.

It is recommended that, as a field concrete shrinkage test, ASTM adopt a cylindrical specimen that is kept in its cardboard mold until the mold is stripped at the start of shrinkage measurements (Ytterberg 1987). The shrinkage specimen should be cast in a 4- by 8-in. (102- by 203-mm) standard cylindrical mold. In its cardboard mold, the specimen can be supported and insulated until the time of the first length measurement by two polyethylene end spacers that hold it inside a standard 6- by 12-in. (152- by 305-mm) cylindrical cardboard mold. A 4- by 8-in. (102- by 203-mm) cylindrical specimen size, rather than Bazant's 83-mm (3.27-in.)-diam cylindrical specimen is recommended to test concrete containing 1-1/2-in. (38-mm) maximum size aggregate. The larger 4-in. (102-mm)-diam sample will take about 6 weeks, however, rather than 3 weeks, to give data with 95-percent predictive confidence.

Kraai (1982) states that cement is responsible for 82 percent of the drying shrinkage. Therefore, cement mortar shrinkage testing is more meaningful than concrete shrinkage testing. He showed that concrete shrinkage varied from 0.03 to 0.06 percent for the same concrete mix made with constant aggregate and constant cement brand over long periods. He theorized that the variations were due to differences in the cement, with shrinkage increasing in the winter months when cement clinker was stored outside for longer periods.

A report (California Producers Committee on Volume Change) makes some suggestions for modifying the ASTM C 157 concrete shrinkage test. These suggestions can also be applied to the suggested 4- by 8-in. (102- by 203-mm) shrinkage specimen:

*a.* Procedures must be standardized for sampling, mixing, molding, initial storage conditions, and transportation of shrinkage samples to the laboratory. Reference should be made to the ASTM C 172 (ASTM 1994f) method of sampling fresh concrete, which requires compositing a sample from three parts of a truck mixer.

*b.* In field shrinkage tests it will be difficult to comply with the usual storage requirements for a laboratory test. It may be adequate to use the storage conditions of ASTM C 31 (ASTM 1994a) which requires temperatures of 60 to 80 °F (16 to 27 °C) and suggests methods of producing the required temperature and humidity. An outer 6- by 12-in. (152- by 305-mm) cylindrical cardboard mold around a 4- by 8-in. (102- by 203-mm) cylindrical cardboard mold containing the specimen may be sufficient to maintain the proper temperature and humidity.

*c.* In view of the variability of field test conditions, a coefficient of variation of 20 percent is suggested in the interpretation of field shrinkage test results. Also, a limit of 120 percent of average preconstruction shrinkage results is suggested as a limit on field test results. The 20-percent coefficient of variation means that 21.1 percent of the tests will exceed the 120 percent shrinkage limitation simply because of test procedure variables.

## Conclusions

Based on the results of this study, it can be concluded that unrestrained drying shrinkage is most commonly measured by casting a standard size of test prism, demolding 24 hr after casting, and monitoring the length change of the prism stored in standard conditions. Prism size and standard conditions vary considerably according to the different standards of different countries (Table 20).

**Table 20**
**Unrestrained Shrinkage Standard Test Methods**

| Country | Standard/ Specification | Prism Dimensions mm | Prism Dimensions in. | Specified Conditions | References |
|---|---|---|---|---|---|
| United States | ASTM C 157 | 25.0 × 25.0 × 285 | 1.0 × 1.0 × 11.0 | 23 C (73 F), 50% RH | ASTM (1991c) |
| United Kingdom | BS 1881, Part 5-1970 | 75.0 × 75.0 × 310 | 3.0 × 3.0 × 12.0 | This standard does not relate to drying shrinkage. Therefore, test conditions are not included. | UK (1970) |
| Canada (Alberta Transportation & Utilities) | ATU B-391 | 25.0 × 25.0 × 285 | 1.0 × 1.0 × 11.0 | Initial 24 hr curing per manufacturer's recommendations. After initial curing: 23 C (73 F), 50% RH | ATU (1992) |
| Australia | AS 1012 Part 3-1970 | 75.0 × 75.0 × 285 | 3.0 × 3.0 × 11.0 | 23 C (73 F), 50% RH | Australia (1970) |
| Germany | DIN 52450-1985 | 37.5 × 37.5 × 155 | 1.5 × 1.5 × 6 | Various-<br>20 C (68 F), 65% RH<br>23 C (73 F), 50% RH<br>23 C (68 F), 45% RH<br>20 C (68 F), 95% RH<br>20 C, (68 F), Wet | Germany (1985) |
| Hong Kong | HKHA | 25.0 × 25.0 × 285 | 1.0 × 1.0 × 11.0 | 25 C (81 F), 55% RH | Hong Kong Housing Department (1987) |

Note: Reference information listed in section following main text.

Although various research studies have been reported on the free shrinkage of the concrete and other materials and the factors influencing the shrinkage, the stresses induced when this shrinkage is restrained and the consequent risk of cracking are of greater importance in repaired concrete structures. In practice, it is seldom possible for repair to shrink freely because restraints of shrinkage are present internally and externally. Restraint may be due to a variety of factors, including monolithic construction, the presence of adjacent structural members, steel reinforcement, or a different moisture gradient, either between the surfaces of a structural member or between the core and the surfaces of the member. It is the strain induced by the restrained shrinkage rather than the magnitude of the "free shrinkage" that leads to cracking. As a result of the restraint, cracking occurs when the induced tensile strain, relieved by creep, exceeds the tensile strain capacity of the material. Such cracking often leads to the corrosion of steel reinforcement and thus jeopardizes structural stability and/or durability.

Despite the obvious value of measuring the effects of restrained shrinkage, as opposed to those of unrestrained or free shrinkage, there is only a limited amount of published data on this topic.

There are several test methods for testing the restrained shrinkage. However, these methods give only quantitative results (amount and width of cracks) but do not demonstrate the quantitative properties (stress developed).

Very few tests, if any, to date, have incorporated stress or cyclic stress on the specimen concurrent with exposure to the environment. Geometry (the size and thickness of the surface repair) is an important parameter but cannot be scaled linearly with respect to the prototype. Investigators rarely attempt to relate the geometry to the degree of shrinkage quantitatively but often draw conclusions from tests which are not properly related to the prototype. In many cases, the level of stress and cyclic loading will affect the initiation, growth, and distribution of cracks in the repair, thus influencing subsequent long-term performance.

Large variations in field performance and in laboratory test results have been observed in what are purported to be similar structures or test specimens and under what were purported to be similar conditions. Therefore, planning field and laboratory tests for statistical validity and reliability is particularly important.

When test procedures are planned in relationship to well-defined criteria and performance requirements, the results of such tests can be interpreted with greater clarity and can lead to significant conclusions. Often, test results have not been planned in accordance with the preceding considerations in mind. Many of the tests reported in the literature have specific and narrow objectives. Interpretation of such tests with respect to their general validity and significance may be questionable.

A general review of various test methods for evaluation of shrinkage of cement-based materials leads to the following conclusions:

*a.* There are no standardized or widely accepted test methods to evaluate drying shrinkage in repairs exposed to real-life environment. Existing test methods are unsatisfactory for predicting the field performance of surface repairs.

*b.* Many tests have been reported without reference to performance requirements or criteria. Interpretation of such tests is often controversial.

*c.* The test methods in different codes of practice promote a narrow view of a complex problem - shrinkage and the factors affecting shrinkage. In our view, the shrinkage test methods presently used are part of this problem rather than part of the solution.

## Recommendations

*a.* Since the results from the existing shrinkage tests cannot be used in predicting the performance of field repair systems, more laboratory and field studies are needed to confirm whether the test methods selected as most promising can be used as a reliable shrinkage test methods.

The following existing test methods are recommended for further studies:

- ASTM C 157, Modified, by Alberta (ATU 1987, 1992)
- Shrinkage Cracking Ring Method (Swamy, Bandyopadhyay, and Stavrides 1979)
- German Angle Method (German Federal Ministry for Transport, Highway Construction Department 1990)

*b.* As an addition to the scope of this project subtask, the "restrained volume change strain/stress indicator" (Figures 36 and 37) is designed based on the results of the preliminary experiments and in consideration of several variables which are, at least initially, considered important for determining volume changes of cement-based materials, including shrinkage. This test reflects the true dimensional behavior of repair systems and allows for simulation of the physical effects responsible for distress in real world repairs. This method is recommended for the studies in Phase II of the project.

*c.* The recommended reliable shrinkage test method (or methods) should account for the four major factors: restrain conditions, geometry, loading, and environment.

*d.* Laboratory and field tests in the experimental program should be planned for statistical validity and reliability.

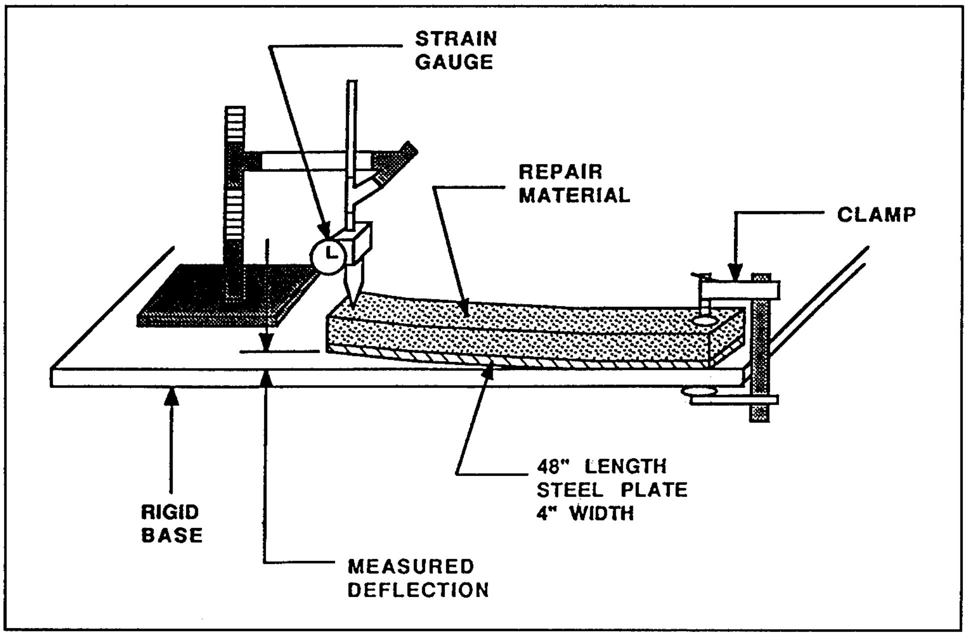


Figure 36. Restrained volume change strain/stress indicator (SPS Plate Test). General view

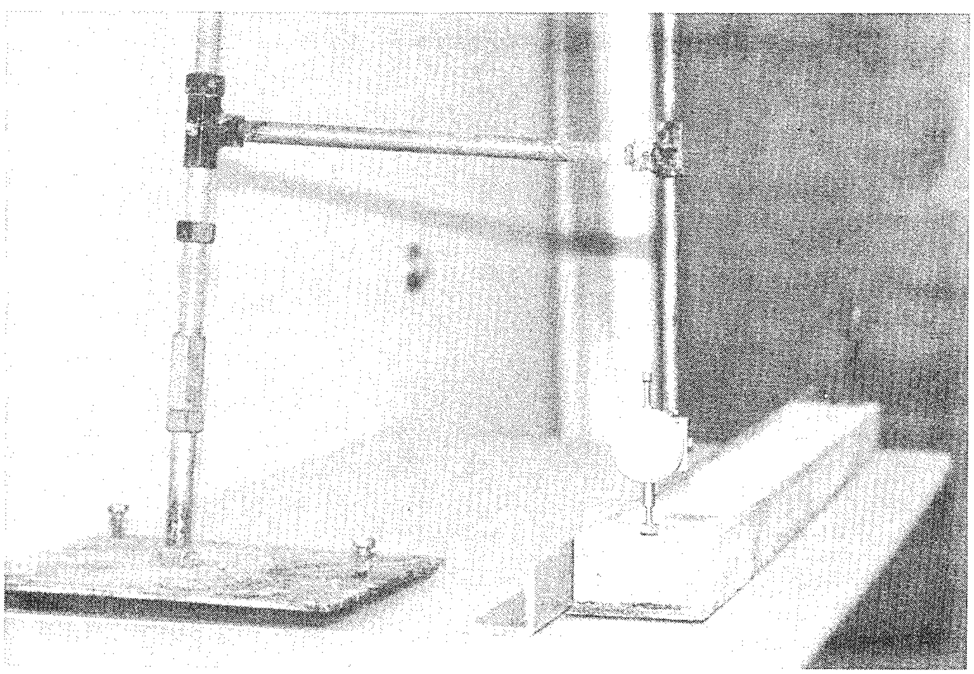

Figure 37. Restrained volume change strain/stress indicator (SPS Plate Test). Shrinkage strain testing

# 5 Performance Criteria

## Previous Studies on Drying Shrinkage of Repair Materials

The comprehensive review of all information obtained from the various sources, together with the analysis of the results of the literature review and survey of selected research and evaluation studies, lead to the conclusion that a very limited amount of work was done on the evaluation of repair materials for practical applications.

The following is a short review of studies of surface repair materials in general and the drying shrinkage property of materials in particular.

### Alberta Concrete Patch Evaluation Program (ACPEP) (ATU 1987)

The study noted some suspected failure mechanisms for bridge deck patches as follows:

*a.* Debonding due to plastic or long-term shrinkage of the patch.

*b.* Debonding due to differences in the coefficient of thermal expansion between the patch and the substrate.

*c.* Debonding due to the buildup of vapor pressure in the substrate when the patch has poor vapor transmission ability.

*d.* Debonding due to deterioration of the substrate caused by moisture reaching a nonair-entrained substrate through a cracked or porous patch material.

*e.* Freeze-thaw deterioration of the patch.

*f.* Cracking, crushing, or abrasion damage due to poor product properties of the patch.

*g.* Spalling due to rebar corrosion resulting from the chemistry of the patch.

*h.* Spalling from rebar corrosion below the patch due to low pH, low electrical resistance, or salt intrusion.

The Alberta Test Program attempted to develop guidelines for selecting the standard tests most important in predicting the performance of patching products. Some of the questions addressed by ACPEP are listed below:

*a.* Are some products more forgiving than others when less than optimum workmanship is used to apply the patches?

*b.* How important are surface preparation, depth, and edge shape of the cavity on the performance of patching products?

*c.* Which physical properties of a concrete patching product are most critical to the service life and field performance of that product?

*d.* How do various commercial products including local materials compare with each other in their field performance?

*e.* Can standard test methods be employed to judge or relatively compare the products?

*f.* Is it possible to assign a relative service life to a repaired concrete patch to reflect the type of patch, surface preparation, exposure conditions, and quality of patching products?

Forty-six proposed products were tested for shrinkage based on data submitted by the manufacturers as well as references from various users.

All products were mixed and cured as per the manufacturers' directions. When the manufacturer recommended that no curing was required unless it was very hot and dry and windy, the specimens were stored at normal laboratory temperature and humidity. If the manufacturer required damp curing, then the materials were placed in the moist room for 24 hr. No curing compounds were used.

The patching products were tested as outlined in ASTM C 157, "Standard test method for length change of hardened-hydraulic cement mortar and concrete," with the following modifications:

*a.* Length change measurements were taken at 24 hr, 3 days, 28 days, and at 60 days from the day of casting.

*b.* The specimens were cured as previously described, then stored at standard laboratory temperature and humidity.

Alberta Shrinkage Test results are presented in Table 21.

**Table 21**
**Alberta Shrinkage Information from Table No. B3 of Alberta Report (ATU 1987)**

| Product No. | Shrinkage 28 days, % |
|---|---|
| 1 | 0.177 |
| 2 | 0.065 |
| 3 | 0.147 |
| 4 | 0.279 |
| 5 | 0.212 |
| 6 | 0.265 |
| 7 | 0.249 |
| 8 | 0.078 |
| 9 | 0.130 |
| 10 | 0.088 |
| 11 | 0.100 |
| 12 | 0.053 |
| 13 | 0.204 |
| 14 | 0.100 |
| 15 | 0.112 |
| 16 | 0.075 |
| 17 | 0.155 |
| 18 | 0.100 |
| 19 | 0.097 |
| 20 | 0.265 |
| 21 | 0.086 |
| 22 | 0.028 |
| 23 | 0.185 |
| 24 | 0.271 |
| 25 | 0.083 |
| 26 | 0.003 |
| 27 | 0.082 |
| 28 | 0.095 |
| 29 | 0.026 |
| 30 | 0.183 |
| 31 | 0.065 |
| 32 | 0.216 |
| 33 | 0.129 |
| 34 | 0.247 |
| 35 | 0.309 |
| 36 | 0.178 |
| 37 | 0.079 |
| 38 | 0.065 |
| 39 | 0.031 |
| 40 | -0.009 |
| 41 | 0.210 |
| 42 | 0.059 |
| 43 | 0.048 |
| 44 | 0.253 |
| 45 | 0.107 |
| 46 | 0.022 |

### Alberta evaluation of length change of concrete patching materials to ATU Specification B-391 (ATU 1992)

Alberta Transportation and Utilities developed Specification B-391, "Specification for evaluation of length change of bridge concrete patching materials" (ATU 1992). This document established minimum requirements of the properties for patching materials used in the repair of concrete bridges in Alberta, including shrinkage. Thirteen materials were evaluated in accordance with B-391.

To provide an evaluation of the patching materials in conditions that more closely represent field conditions, the following changes to the ASTM C 157 standard have been incorporated into the ATU Specification B-391 (ATU 1992).

*a.* Length changes are referenced to the gage length of the prism at the time of casting. Length change determinations are required at ages of 1, 3, 7, 28, 60, and 120 days.

*b.* Curing of the specimens has been altered to reflect the curing conditions which could be encountered during the patching of bridges. During favorable patching conditions, it is expected that all the manufacturers' instructions will be followed. Therefore, one set of specimens is required to be prepared using the manufacturers' recommendations for curing during the initial 24 hr to represent favorable curing conditions. However, it is also expected that there could be some circumstances when the manufacturers' instructions may not or cannot be followed. A second set of specimens is required to determine the effect on length change if the manufacturers' recommendations are not followed. This second set of specimens is cured at 50-percent relative humidity for the initial 24 hr after casting and at a relative humidity of 50 percent for the remainder of the test period.

The objective of this study was to evaluate the length change characteristics of 13 prebagged patching materials based on the ATU Specification B-391 (ATU 1992) revisions to the ASTM Test Method C 157. Shrinkage test results are presented in Table 22.

### Hydro-Quebec evaluation program (Mirza and Durand 1993)

Hydro-Quebec initiated a joint Canadian Electrical Association (CEA) project and studied 40 mortars from different manufacturers. The main objectives of Phase I were to modify and optimize the properties of repair materials, and to test newly marketed products. Products showing a better performance than the normal cement mortar (used as the reference) were then subjected to small scale tests (Phase II).

**Table 22**
**Length Change in Accordance with Specification B-391, Table 1 (ATU 1992)**

| Mat No. | Initial 24-hr Curing Conditions | Length Change (%)[1] | | | | | |
|---|---|---|---|---|---|---|---|
| | | 1 Day | 3 Days | 7 Days | 28 Days | 60 Days | 120 Days |
| | ATU specifications 50% RH | 0.051 | 0.054 | 0.057 | 0.071 | 0.074 | 0.084 |
| 1. | Manufacturer recommendations air-cured 39% RH[2] | 0.050 | 0.053 | 0.058 | 0.070 | 0.073 | 0.079 |
| | ATU specifications 50% RH | 0.010 | 0.033 | 0.064 | 0.087 | 0.097 | 0.098 |
| 2. | Manufacturer recommendations air-cured 36% RH[2] | -0.026 | 0.014 | 0.016 | 0.037 | 0.046 | 0.053 |
| | ATU specifications 50% RH | 0.107 | 0.165 | 0.185 | 0.200 | 0.204 | 0.213 |
| 3. | Manufacturer recommendations air-cured 22% RH[2] | 0.058 | 0.107 | 0.127 | 0.149 | 0.154 | 0.163 |
| | ATU specifications 48%RH | 0.037 | 0.101 | 0.132 | 0.144 | 0.155 | 0.171 |
| 4. | Manufacturer recommendations air-cured 22% RH[2] | 0.027 | 0.085 | 0.117 | 0.132 | 0.145 | 0.159 |
| | ATU specifications 46% RH | 0.018 | 0.043 | 0.047 | 0.048 | 0.058 | 0.063 |
| 5. | Manufacturer recommendations air-cured 30% RH[2] | 0.014 | 0.043 | 0.046 | 0.048 | 0.056 | 0.063 |
| | ATU specifications 48% RH | 0.124 | 0.213 | 0.231 | 0.241 | 0.243 | 0.254 |
| 6. | Manufacturer recommendations 100% RH | 0.032 | 0.196 | 0.229 | 0.240 | 0.240 | 0.252 |
| | ATU specifications 48% RH | 0.068 | 0.081 | 0.089 | 0.093 | 0.104 | 0.115 |
| 7. | Manufacturer recommendations 100% RH | 0.027 | 0.040 | 0.049 | 0.056 | 0.065 | 0.078 |
| | ATU specifications 48% RH | -0.005 | 0.014 | 0.016 | 0.016 | 0.019 | 0.019 |
| 8. | Manufacturer recommendations 100% RH | -0.003 | 0.029 | 0.033 | 0.033 | 0.034 | 0.035 |
| | ATU specifications 50% RH | -0.018 | -0.001 | 0.004 | 0.006 | 0.003 | 0.003 |
| 9. | Manufacturer recommendations 100% RH | -0.029 | 0.004 | 0.013 | 0.014 | 0.014 | 0.015 |
| | ATU specifications 50% RH | -0.006 | 0.000 | 0.010 | 0.022 | 0.026 | 0.032 |
| 10. | Manufacturer recommendations air-cured 15% RH[2] | -0.065 | -0.060 | -0.050 | -0.038 | -0.034 | 0.029 |
| | ATU specifications 50% RH | -0.009 | 0.006 | 0.015 | 0.042 | 0.048 | 0.056 |
| 11. | Manufacturer recommendations 50% RH | 0.002 | 0.015 | 0.033 | 0.053 | 0.058 | 0.067 |
| | ATU specifications 50% RH | 0.031 | -0.038 | -0.040 | -0.032 | -0.025 | -0.016 |
| 12. | Manufacturer recommendations 50% RH | -0.031 | -0.038 | -0.038 | -0.030 | -0.026 | 0.009 |
| | ATU specifications 50% RH | 0.039 | 0.087 | 0.127 | 0.171 | 0.189 | 0.216 |
| 13. | Manufacturer recommendations air-cured 16% RH[2] | 0.030 | 0.073 | 0.113 | 0.156 | 0.177 | 0.207 |

[1] Length change determination in accordance with ASTM C 157 (ASTM 1994e) and as modified by Alberta Transportation and Utilities B-391 (ATU 1992). All specimens cured in air at 50% R after initial 24-hr cure.
[2] Measured relative humidity.

A total of 40 mixes (Table 23) taken from 37 repair materials from 13 different manufacturers were selected for this study. These materials may be grouped into four categories: cement mortars, including ordinary cement mortar, cement mortar containing silica fume and aluminous cement mortar; polymer-modified cement-based mortars containing styrene-butadiene rubber (SBR) and acrylics; sand-epoxy mortars and emulsified epoxy mortars. A mix (Cl) consisting of normal portland cement with a w/c of 0.46 and sand-cement ratio of 2.75 was used as a reference throughout the study. The test data summary of the laboratory-tested products is presented in Table 24.

## Classification of Repair Materials

To compare the 46 materials tested in the Alberta Concrete Patch Evaluation Program, shrinkage test results were sorted from low shrinkage to high shrinkage. Figure 38 presents a diagram of the results of the shrinkage testing sorted from low shrinkage on the left to high shrinkage on the right.

For purposes of developing a preliminary performance criteria, we propose a classification of repair materials based on their 28-day drying shrinkage properties tested in accordance with ASTM C 157 (Modified by Alberta). As a benchmark, shrinkage of normal-weight concrete at 28 days - 0.05 percent (Troxell, Raphael, and Davis 1958) is taken. The repair materials are grouped into four basic categories of shrinkage: very low, low, moderate, and high (Figure 39). Any material showing a shrinkage less than 0.025 percent is regarded as very low shrinkage, a material showing shrinkage from 0.025 percent to less than 0.05 percent is categorized as low shrinkage, materials with shrinkage of 0.05 percent through 0.10 percent (two times the shrinkage of plain concrete) have been categorized as moderate shrinkage, and materials with shrinkage in excess of 0.10 percent have been labeled as high shrinkage.

If we assume that 46 materials represent a cross section of North America's industry of repair materials, then the following conclusions can be made:

*a.* Only 6.5 percent of materials tested have shrinkage of less than 0.025 percent and can be labelled as very low-shrinkage materials.

*b.* Only 15 percent of the materials tested have shrinkage less than concrete (0.05 percent) and can be labeled as low-shrinkage materials.

*c.* At least 85 percent of the materials tested have shrinkage higher than concrete.

*d.* At least 54 percent of the materials tested have shrinkage in excess of 0.10 percent and can be labeled as high-shrinkage materials.

## Table 23
## Description of the Surface Repair Materials Tested in the Laboratory (Mirza and Durand 1993)

| Mixes | Type of Materials | Description |
|---|---|---|
| C1 | Cement mortars/cement grouts | Type 10 cement + Ottawa sand (reference mortar) |
| C2 | | Type 10 cement + Ottawa sand + superplastisizer[1] |
| C3 | | Type 10 cement + 6% silica fume + sand + superplastisizer[1] |
| C4 | | Type 10 cement + 9% silica fume + sand + superplastisizer[1] |
| C5 | | Type 10 cement + 12% silica fume + sand + superplastisizer[1] |
| C6 | | Type 10 cement + 15% silica fume + sand + superplastisizer[1] |
| C7 | | Inorganic cements + additives + abrasion resistant aggregates |
| C8 | | Portland cement + fines + additives + nonshrinkage agent |
| C9 | | Type 10 cement + graded sand + nonferrous admixtures |
| C10 | | Cement and premixed sand + additives |
| C11 | | Cement and premixed sand + additives |
| C12 | | Cement and premixed sand + additives + fibers |
| C13 | | Type 10 cement + sand + gassing and flowing agent |
| C14 | | Aluminous cement + artificial aggregates |
| C15 | | Aluminoius cement + bonding agent + artificial aggregates |
| R16 | Styrene butadiene rubber (SBR) cement-based mortars | Latex emulsion + premixed cement-based powder + additives |
| R17 | | Latex emulsion + premixed cement-based powder + additives |
| R18 | | Same as R17 + glass reinforced fibers |
| R19 | | Latex emulsion with water + premixed cement-based powder + additives[2] |
| | | Latex emulsion + cement + graded aggregates[2] |
| R20 | | Latex emulsion + cement + premixed graded aggregates |
| R21 | | Latex emulsion + Portland cement-based ingredients |
| R22 | | |
| A23 | Acrylic cement-based mortars | Latex emulsion + premixed nonshrinkage cement-based powder + low-density silica fume |
| A24 | | Latex emulsion + premixed cement-based powder + additives |
| A25 | | Latex emulsion + premixed cement-based powder + additives |
| A26 | | Latex emulsion + premixed cement-based powder + additives |
| E27 | Sand epoxy mortars | Resin + hardener + premixed graded aggregates |
| E28 | | Resin + hardener + graded sand |
| E29 | | Resin + hardener + graded aggregates |
| E30 | | Low modulus resin + hardener + graded sand[3] |
| E31 | | High modulus resin + hardener + graded sand[3] |
| E32 | | High modulus resin + hardener + graded sand |
| E33 | | Low modulus resin + hardener + graded sand |
| E34 | | Resin + hardener + silica sand[3] |
| E35 | | Resin + hardener + premixed graded aggregates |
| E36 | | Resin + hardener + graded sand |
| E37 | | Resin + hardener + graded sand |
| E38 | | Resin + hardener + quartz aggregate |
| EE39 | Emulsified epoxy mortars | Resin + hardener + sand + cement |
| EE40 | | Resin + hardener + sand + cement |

[1] In-house mortar (with Ottawa sand, Illinois, USA).
[2] Mortars fabricated with the manufacturers' assistance.
[3] Mortars selected and modified from our previous study.

## Table 24
## Test Data Summary of the Laboratory-Tested Products (Mirza and Durand 1993)

| Product Code | Bond Strength 28-day MPa Dry Cure | Bond Strength 28-day MPa Wet Cure | Coeff. of Abrasion Erosion | Drying-Shrinkage % Net Change | Drying-Shrinkage % Total Change | Comp Strength 28-day MPa Dry Cure | Comp Strength 28-day MPa Wet Cure | Coeff. of Thermal Exp. $\times 10^{-6}$ | Thermal Compat. % Disbonded | Coeff. of Permeability Kc (cm/s) $\times 10^{-9}$ |
|---|---|---|---|---|---|---|---|---|---|---|
| C1 | 22.3 | 15.3 | 1.32 | -0.097 | 0.123 | 31.9 | 29.0 | 10.2 | 35 | 10.8 |
| C2 | 15.1 | 26.2 | 2.06 | -0.14 | 0.238 | 33.5 | 43.4 | 9.9 | 10 | 77.5 |
| C3 | 26.8 | 30.3 | 1.63 | -0.214 | 0.180 | 45.9 | 51.1 | 9.9 | 5 | 3.2 |
| C4 | 29.9 | 45.4 | 1.42 | -0.166 | 0.226 | 46.8 | 53.8 | 9.8 | 0 | - |
| C5 | 29 | 39.8 | 1.95 | -0.171 | 0.221 | 50.3 | 61.0 | 9.9 | 0 | - |
| C6 | 37.9 | 41.0 | 2.38 | -0.16 | 0.229 | 49.4 | 61.1 | 10.1 | 0 | - |
| C7 | 44.9 | 37.4 | | -0.067 | 0.165 | 72.8 | 73.3 | 12.9 | 20 | 18.6 |
| C8 | 50.2 | 47.3 | 2.17 | -0.076 | 0.163 | 79.6 | 73.5 | 10.9 | 100 | 1.9 |
| C9 | 24.1 | 43.7 | 3.28 | -0.078 | 0.118 | 41.9 | 44.0 | 9.7 | 0 | 9.6 |
| C10 | 39.8 | 33.5 | | -0.123 | 0.174 | 52.1 | 70.9 | 9 | 10 | 1.5 |
| C11 | 34 | 62.3 | 2.36 | -0.154 | 0.174 | 59 | 77.3 | 9.7 | 5 | 4.9 |
| C12 | 33.8 | 37.8 | 1.94 | -0.105 | 0.118 | 58.6 | 65.3 | 9.6 | 100 | 51.8 |
| C13 | 36.9 | 47.3 | 2.4 | -0.089 | 0.125 | 48.8 | 52.1 | 7.4 | 95 | 9.4 |
| C14 | 28.6 | 54.9 | 4.45 | -0.02 | 0.050 | 82.1 | 95.1 | - | 5 | 9.9 |
| C15 | 50.4 | 64.8 | 3.23 | -0.037 | 0.089 | 85.3 | 115.2 | 7.3 | 85 | 10.5 |
| R16 | 34.7 | 36.4 | 2.51 | -0.005 | 0.091 | 57.8 | 54.8 | 14 | 95 | 0.1 |
| R17 | 34.1 | 26.4 | 2.57 | 0.086 | 0.433 | 48.1 | 46.5 | 11.5 | 95 | 1.6 |
| R18 | 31.3 | 25.3 | 3.32 | 0.02 | 0.433 | 45.1 | 46.0 | 12.1 | - | 0.39 |
| R19 | 49.9 | 37.7 | 3.3 | -0.001 | 0.109 | 62.6 | 61.5 | - | 0 | - |
| R20 | 68.7 | 60.4 | 3.32 | -0.143 | 0.171 | 85.9 | 80.5 | 7.7 | 0 | 41.9 |
| R21 | 38.1 | 34.2 | 2.82 | -0.179 | 0.201 | - | 53.2 | 7.9 | 0 | 2.4 |
| R22 | 16.6 | 10.2 | 0.92 | -0.063 | 0.087 | 23.2 | 24.4 | 10.4 | - | 1.9 |
| A23 | 16.5 | 20.4 | 2.1 | -0.096 | 0.180 | 35.6 | 27.3 | 11.8 | - | 0.28 |
| A24 | 28.4 | 17.8 | 2.6 | -0.092 | 0.163 | 52.3 | 50.2 | 11.7 | 0 | 1.1 |
| A25 | 32.8 | 32.6 | 3.11 | 0.052 | 0.253 | 55 | 44.2 | 12.5 | 0 | 0.08 |
| A26 | 33.6 | 35.1 | 1.64 | -0.015 | 0.193 | 44.8 | 27.7 | 12.4 | 70 | 0.1 |
| E27 | 2.6 | - | 3.97 | 0.094 | 0.619 | 55.8 | - | 18.7 | 90 | >10000 |
| E28 | 0.7 | - | 3.28 | 0.081 | 0.131 | 27.5 | - | - | - | >10000 |
| E29 | 31.0? | - | 5.4 | 0.131 | 0.128 | 44.2 | - | 23.1 | 10 | 18.9 |
| E30 | 3.4 | - | 1.88 | 0.165 | 0.295 | 36.9 | - | - | - | >10000 |
| E31 | 4.4 | - | 2.68 | 0.135 | 0.279 | 30.2 | - | - | - | >10000 |
| E32 | 5.1 | - | 2.12 | 1.065 | 1.431 | 36.7 | - | - | - | >10000 |
| E33 | 4.2 | - | 4.81 | 0.316 | 0.337 | 37.7 | - | - | - | >10000 |
| E34 | 0.3 | - | 0.63 | 0.029 | 0.03 | 16.1 | - | - | - | - |
| E35 | 18.3 | - | 5.22 | -0.07 | 0.071 | 68.8 | - | 20 | 0 | 184 |
| E36 | 10 | - | 3.17 | 0.003 | 0.106 | 40 | - | 22.4 | - | >10000 |
| E37 | 22.6 | - | 5.35 | 0.011 | 0.026 | 73.8 | - | 19.5 | 100 | 437 |
| EE39 | 7.2 | 11.8 | 0.87 | -0.086 | 0.098 | 20.1 | 21.8 | - | - | - |
| EE40 | - | 8 | - | - | - | 27.9 | 14.4 | - | - | - |

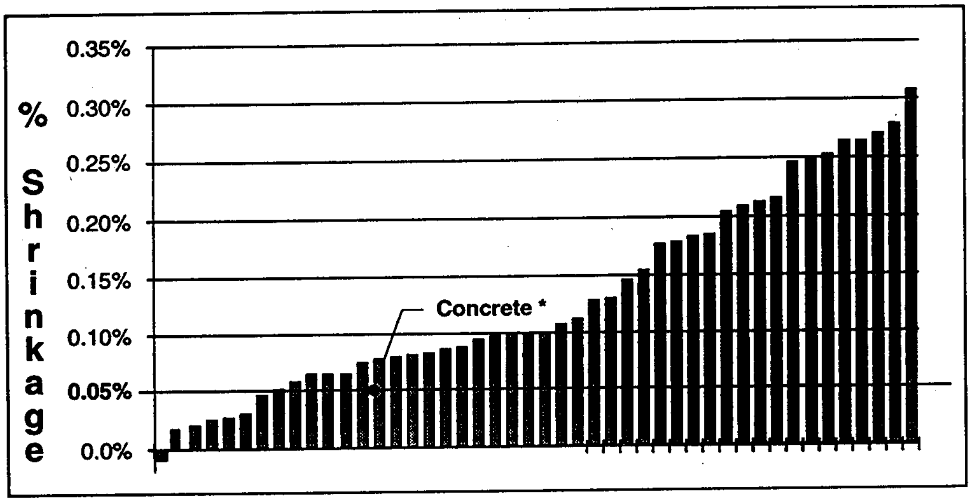


Figure 38. Alberta shrinkage test results

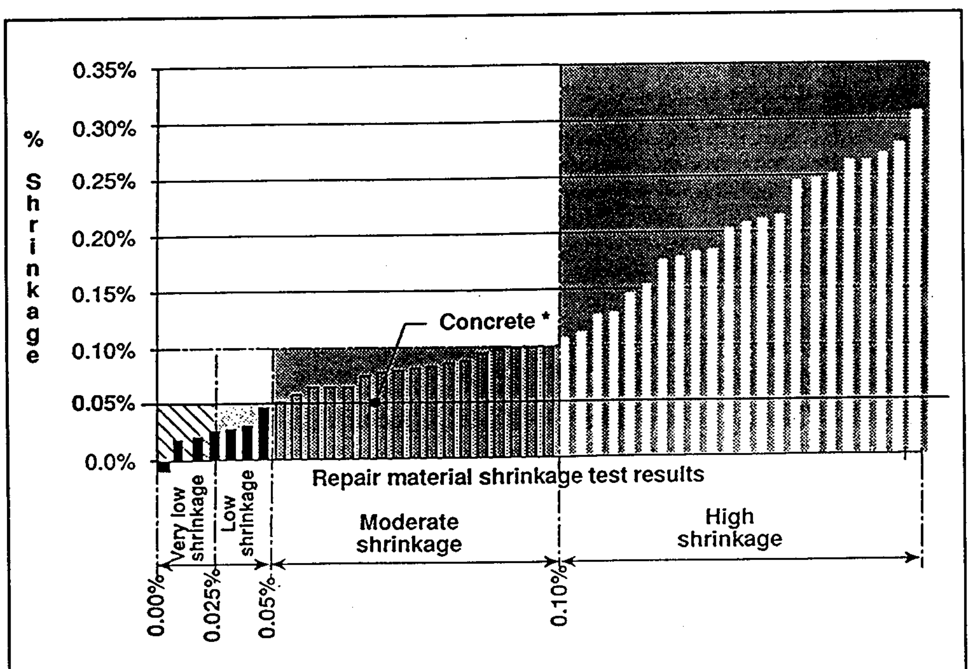


Figure 39. Classification of repair material based on drying shrinkage

Based on the proposed classification, 13 rapid patch materials tested by Alberta Transportation and Utilities in accordance with Alberta Specification B-391 (ATU 1992) were also analyzed. The results of the analysis are presented in Figure 40.

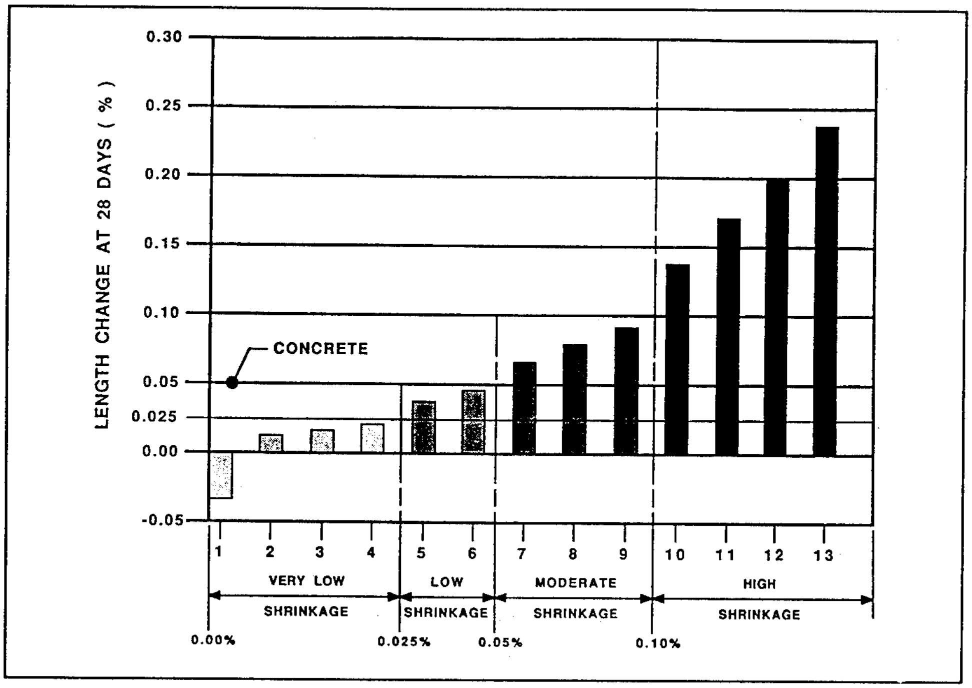


Figure 40. Classification of repair materials

With regard to the factors influencing stress in repair material and its resistance to cracking, we found the formula presented by Conproco (Brown 1991) to be of special interest:

$$P = T - 33\text{Ø}^{3/2}\sqrt{f^1{}_c}\; e_{sh}\; R_{dp}$$

where

P = "reserve" number (psi)

T = tensile strength (psi) ASTM C 190 (ASTM 1994g)

Ø = unit weight

$f^1{}_c$ = compressive strength (psi), ASTM C 109 (ASTM 1994d)

$e_{sh}$ = shrinkage factor (in./in.) ASTM C 157 (ASTM 1994e)

$^1R_{dp}$ = stress reduction factor (unitless)

According to Conproco (Brown 1991), this formula allows a prediction of the cracking potential of a repair material. This formula addresses the relative importance of the shrinkage stress reduction factors, based on the calculation of creep and modulus of elasticity of the repair material. This explains the phenomena that material with high free shrinkage will not crack if it can relax the stress through creep.

Another important factor which is considered is the tensile strength of the repair material as a critical item for cracking, especially during the first week after application of the repair.

The formula has certain limitations. According to Conproco (Brown 1991);

*a.* "the values of properties of the repair materials must all be for the same cure period. A look at the probability of cracking at 3 or 7 days is done through two separate calculations. One done with *all* physicals at 3 days and another done with *all* physicals at 7 days. By using this technique, a graph can be developed. This graph may be useful in predicting how long it will take for cracks to appear in the patch material.

*b.* "the values of properties of the repair materials must be determined by a test which simulates the actual field condition. For example, if looking at bond strength, ASTM C 882 (ASTM 1994s), a compressive slant shear bond test is not indicative of "real world" bond line stresses induced by shrinkage. A test like ASTM C 932 (ASTM 1994v), a tensile bond test, is more representative and will produce better calculated to actual performance correlations.

We have found the following tests to correlate well.

| | ASTM Procedure |
|---|---|
| Tensile Strength | C 190 (ASTM 1994g) |
| Compressive Strength | C 39 or C 109 (ASTM 1994b, ASTM 1994d) |
| Bond Strength | C 932 (ASTM 1994v) |
| Shrinkage | C 157 (ASTM 1994e) |

*c.* The modulus of elasticity per the PCA Manual, *Design and Control of Concrete Mixtures*, 11th Edition (PCA 1967) on page 89 is:

$$e = 33\ w\ 3/2\ \sqrt{f'_c}\ (w\ -\ \text{unitweight, pcf})$$

This is good for 3,000 psi to about 6,500 psi materials. This correlation is used since for most materials, the modulus of elasticity is unknown. For more exact results, the modulus of elasticity (e) should be determined by testing. We have found this approximation to work well in our equation for materials up to 7,000+ psi, but when the value of e is known, the known value should be substituted in the 33Ø $^{3/2}\sqrt{f'_c}$ portion of the equation."

The presented formula was evaluated for six materials at the age of 28 days (see Table 25). The data presented in this table allowed Conproco (Brown 1991) to arrive at the following conclusions:

"In regard to some actual results, if you look at the table for 28 days, Set and Structural Skin with 2 quarts of K-88 are the plus side in reference to the "reserve" number (P). This implies they have excess tensile strength to resist the forces induced by shrinkage. This is attributable to the ability to "absorb" the stress and is reflected in the low stress reduction factor ($R_{dp}$). To go to the other extreme, sand-cement mortar is on the "negative" side of the reserve number. This implies the stresses induced on shrinkage have exceeded the physical properties and cracking should result. In both cases, the calculated result agrees with actual exposures. Conproco has had little problem with cracking in Set or Structural Skin with K-88 while field mix sand-cement mortar is known to crack.

**Table 25**
**Material Tests and Evaluation Data (after Brown 1991)**

| Material | T | Ø | $f'_c$ | $e_{sh}$ in./in. | % Shr. | Proposed Classification | $R_{dp}$[1] | P |
|---|---|---|---|---|---|---|---|---|
| Set | 600 | 125.5 | 7048 | 0.0006 | 0.06 | Moderate | 0.2220 | +84.18 |
| Structural Skin | 400 | 113.9 | 4600 | 0.0003 | 0.03 | Low | 0.4560 | +27.90 |
| Structural Skin with 2 qt of K-88 | 635 | 24.5 | 6300 | 0.0005 | 0.05 | Moderate | 0.2720 | +140.0 |
| Sand/cement Mortar | 200 | 104.6 | 3500 | 0.0005 | 0.05 | Moderate | 0.5820 | -808.2 |
| OS30 (Experimental) | 475 | 120.5 | 6213 | 0.00005 | 0.05 | Moderate | 0.3120 | -61.64 |

[1] $R_{dp}$ is calculated based on creep and modulus of elasticity.

Also listed is an experimental formulation OS30. This was an interesting case. It did not crack during testing in the lab. However, the equation shows a negative reserve number. When placed outdoors on a larger scale, though, it did crack as predicted.

In summary, an absolute scale has not been developed, but in general, a negative number is undesirable. A "gray" area might exist something like 0-25, but that has yet to be determined. Also, shrinkage is important, but of greater importance is the material's ability to absorb this stress. This ability can be calculated by accounting for the creep and modulus of elasticity. In the submitted formula, this is reflected in $R_{dp}$."

Analysis of the proposed formula by Conproco indicates that free shrinkage property by itself does not determine the capacity of the repair system to resist stresses without cracking. Other properties affecting dimensional compatibility (creep, modulus of elasticity) should also be considered. The results of free shrinkage tests may, therefore, be considered only as an indicator of trends.

The Phase II of this study may allow us to establish the relationship between dimensional compatibility properties and actual field behavior of repair materials in repair systems.

## Studies on Performance Criteria

The complex situation of volume changes of cement-based materials in real-world repair systems exposed to different environmental conditions illustrates that simplifications, generalizations, and extrapolations may be very dangerous.

Difficulties with selecting repair materials have resulted not only from the lack of industry-wide reliable testing standards, for shrinkage in particular, but also from the lack of generally accepted performance criteria.

As was shown previously in the analysis of properties affecting dimensional compatibility, the ability of a repair material to resist cracking is dependent upon:

*a.* Degree of restraint

*b.* Magnitude of the shrinkage due to drying and thermal effects

*c.* Stresses produced in the material

*d.* Amount of stress relief due to creep

*e.* Tensile strength of the material

It was demonstrated in this study that to have durable repairs with good resistance to cracking, repair materials should have values of drying shrinkage, coefficient of thermal expansion, and sustained modulus of elasticity as low as possible and a tensile strength as high as possible.

An investigation into the desirable characteristics of repair materials for satisfactory performance was undertaken and the results of this study are summarized in this report.

Traditionally, specifications of concrete repair materials have revolved around compressive strength characteristics. Recently, a changing approach worldwide has meant that much more detailed performance criteria has been specified.

Master Builders Technologies (1993) proposed an approach of re-establishing the equilibrium for structure in restoration. This requires a repair that matches the original structure's physical characteristics and balances the load envelope (Figure 41).

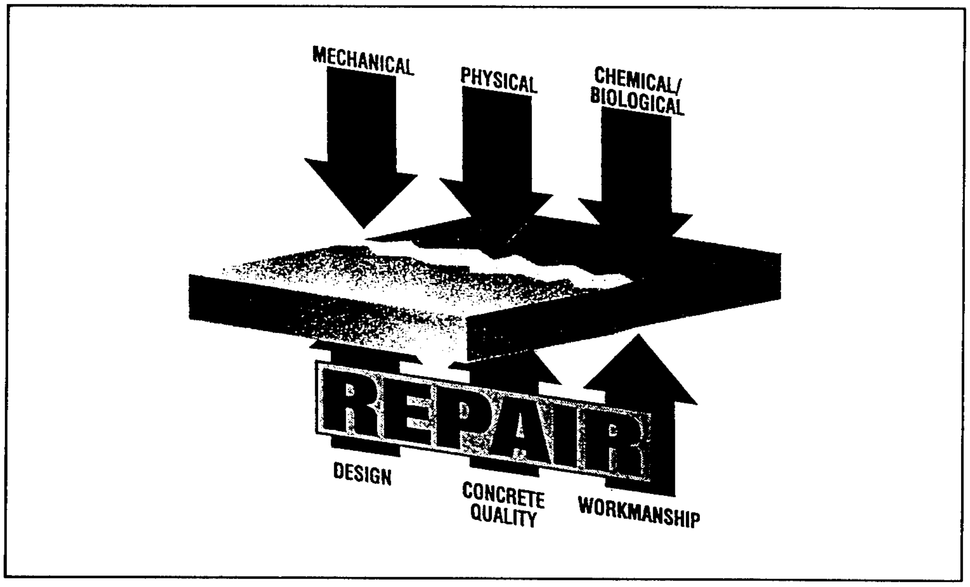


Figure 41. Reinstating equilibrium in restoration (Master Builders Technologies 1993)

Material specifying decisions are based on the ability of the product to work in harmony with the original structure as well as to counteract those factors that caused the deterioration, i.e., loss of equilibrium.

Alberta Transportation and Utilities developed Specification B-391, "Specification for Bridge Concrete Patching Materials" (ATU 1992). This document establishes performance criteria (minimum requirements) for the following properties for patching materials used in the repair of concrete bridges in Alberta:

*a.* Compressive strength

*b.* Shear bond strength

*c.* Water absorption

*d.* Length change

*e.* Resistance to salt scaling

The rating system developed for drying shrinkage in the specifications is presented in Table 26.

**Table 26**
**Rating of Shrinkage Results For All Patching Types**

| Rating | Test Results % |
|---|---|
| 1 | ≥ 0.225 |
| 2 | 0.200 to 0.224 |
| 3 | 0.175 to 0.199 |
| 4 | 0.150 to 0.174 |
| 5 | 0.125 to 0.149 |
| 6 | 0.100 to 0.124 |
| 7 | 0.075 to 0.099 |
| 8 | 0.050 to 0.074 |
| 9 | 0.025 to 0.049 |
| 10 | <0.024 |

Recent Australian Specifications have also included detailed requirements for repair materials including reference to drying shrinkage. The limit of 450 microstrain at 28 days measured to Australian Standard AS 1012 (1970) has been established (Decter and Lambe 1992).

In Hong Kong, the Hong Kong Housing Authority (HKHA) has developed specifications for classes of concrete repair mortars to be used in the repair of concrete structures (Hong Kong Housing Department 1987). This has necessitated the development of specific formulations for prepackaged repair mortars to meet these requirements. The HKHA specification is presented in Table 27 (Hong Kong Housing Department 1987).

In a more recent development, the four mortar classes in the HKHA specification have been rationalized to three broader classes, the highest strength category having been removed. Test Method 8, for drying shrinkage, is also temporarily removed and under review since few materials complied with the specification (Table 28).

## Table 27
## Technical Specifications for Repair Mortars (Hong Kong Housing Department 1987)

**A Characteristics for Repair Mortar**

| Characteristics | | Mortar Class | | | |
|---|---|---|---|---|---|
| | | **60** | **40** | **25** | **15** |
| **TM1** | Range of compressive strength at 28 days MPa (psi approx) | 40-80 (5,800-11,600) | 25-55 (3,600-8,000) | 15-35 (2,200-5,100) | 10-20 (1,500-2,900) |
| **TM2** | Flexural strength | no specific requirement (NSR) | NSR | NSR | NSR |
| **TM3** | Minimum tensile strength MPA (psi approx) | 2.5 (360) | 2.0 (290) | 1.5 (220) | 1.0 (150) |
| **TM4** | Range of modulus of elasticity @28 days GPA (psi approx x $10^3$) | 20-30 (2,900-4,350) | 13-20 (1,900-2,900) | 8-13 (1,160-1,900) | 5-8 (725-1,160) |
| **TM5** | Minimum bond strength at 7 days MPA (psi approx) | 2.5 (360) | 2.0 (290) | 1.5 (220) | 1.0 (150) |
| **TM6** | Cracking in Coutinho Ring Test at 28 days | nil | nil | nil | nil |
| **TM7** | Minimum Figg air permeability (seconds) | 250 | 200 | 300 | 300 |
| **TM8** | Maximum drying shrinkage at 7 days (microstrain = % length change x 10,000) | 300 | 300 | 300 | 300 |

Minimum pull-off stress at 7 days 1/4 × minimum bond strength at 7 days

**B Mix Proporptions for Repair Mortars**

| **Property** | **Permitted Range (measured by weight)** |
|---|---|
| Aggregate cement ratio | 2:1 - 4.5:1 |
| Water cement ratio | 0.3 - 0.5 |
| Minimum polymer solids content by weight of cement: styrene butadiene rubber (SBR) | 8% |
| Acrylic styrene copolymer (ASC) | 5% |
| Polyacrylate copolymer (PAC) | 5% |
| Eethylene vinyl acetate copolymer (EVA) | 5% |

**Notes:**

A The above requirements come from Tables 16.1 and 16.2 of Housing Department's *Manual of Structural Maintenance* (Hong Kong Housing Department 1987).

B Subject to substantiation by the supplier, limit on A/C ratio may be waived for prebagged mortar with lightweight aggregate or filler.

**Table 28**
**Technical Specifications for Repair Mortars (Amended version April 1991) (Chan and Ainsworth 1991)**

| Characteristics for Repair Mortar | | Repair Materials | | |
|---|---|---|---|---|
| **Characteristics** | | **Class 40** | **Class 25** | **Class 15** |
| **TM1** | Range of compressive strength at 28 days MPa (psi approx) | 30-60 (4,350-8,700) | 20-40 (2,900-5,800) | 10-30 (1,450-4,350) |
| **TM3** | Minimum tensile strength at 7 days in MPa (psi approx) | 2.0 (290) | 1.5 (220) | 1.0 (150) |
| **TM4** | Range of modulus of elasticity @ 28 days in KN/mm$^2$ GPa (psi approx) | 15-25 (2,175-3,626) | 9-15 (1,300-2,175) | 5-9 (725-1,300) |
| **TM5** | Minimum bond strength at 7 days in MPa (psi approx) | 2.0 (290) | 1.5 (220) | 1.0 (150) |
| TM6 | Cracking in Coutinho Ring Test at 7 days | 0 | 0 | 0 |
| **TM7** | Minimum Figg air permeability in seconds | 200 | 150 | 100 |

**Notes:**
The repair mortar shall be classified by its range of modulus of elasticity at 28 days.
The consistency of the repair mortar shall suit the actual site conditions. Different consistencies may be required for overhead work, repairs to vertical surfaces, and repairs using soffit board.

Mortar is classified in terms of a range of modulus at 28 days (Chan and Ainsworth 1991). Within each mortar class, an additional seven criteria are laid down. These fall into three broad areas:

*a.* Strength associated characteristics - compressive, flexural, tensile, and bond strengths, and modulus of elasticity.

*b.* Permeability - Figg air permeability.

*c.* Shrinkage - drying shrinkage, Coutinho Ring.

The first two of these areas outline the desirable performance criteria, but the third, i.e., low shrinkage, is imperative if the repair is to remain sound. Strength and permeability characteristics of repair mortars have been specified for some time, but the importance of low shrinkage has only been recognized relatively recently.

The crucial components of a good durable concrete repair mortar encompass compatibility with the substrate, low permeability, and long-term integrity of the repaired area.

The higher temperature (81 °F) and lower relative humidity (55 percent) which make up the standard storage conditions in Hong Kong promote higher drying shrinkage. Figure 42 (Decter and Lambe 1992) shows a plot of drying shrinkage results for a repair material tested under United Kingdom conditions (68 °F, 65 percent RH) cast as a large prism (3- by 3- by 10-1/2 in.) and HKHA conditions (81 °F, 55 percent RH) cast in a small prism (1- by 1- by 11-in.). The large difference in drying shrinkage results illustrates the influence of test specimen size and conditions.

The most stringent drying shrinkage specification is the HKHA TM8, and for this reason results are presented using this as a criterion (Figure 43). This shows the drying shrinkage of the two repair materials in relation to this specification.

Although the proposed classification in this study based on drying shrinkage indicates that certain materials fall in the "very low" and "low" shrinkage categories, these data must be viewed in conjunction with other factors, creep in particular. Unfortunately, performance requirements listed in different studies and specifications are not addressing this property and its effect on repair performance. Creep is a property of great significance when choosing a repair material. Therefore, it will not surprise us if some repair materials failing to perform "adequately" in a free shrinkage test will demonstrate good performance in service and vice versa.

Because of very limited information on the effect of creep on the performance of repair systems, the proposed preliminary performance criteria is not specifying the acceptable limit of this property. However, the study of creep and its effect on overall performance of repair systems is one of the important subtasks of the proposed experimental program.

## Recommended Preliminary Performance Criteria

Based on the results of this study, preliminary performance criteria are given in Table 29. The criteria specify the use of only laboratory cast specimens. The criteria are intended for screening and selecting cement-based surface repair materials subjected to normal environments and loadings. The criteria are *not* intended for special conditions such as underwater repairs or special types of loadings.

Very limited information on laboratory data and field performance were used to develop the preliminary performance criteria in Table 29. The criteria will most likely need to be modified as additional field performance data were obtained as proposed in the following experimental program (Figure 44).

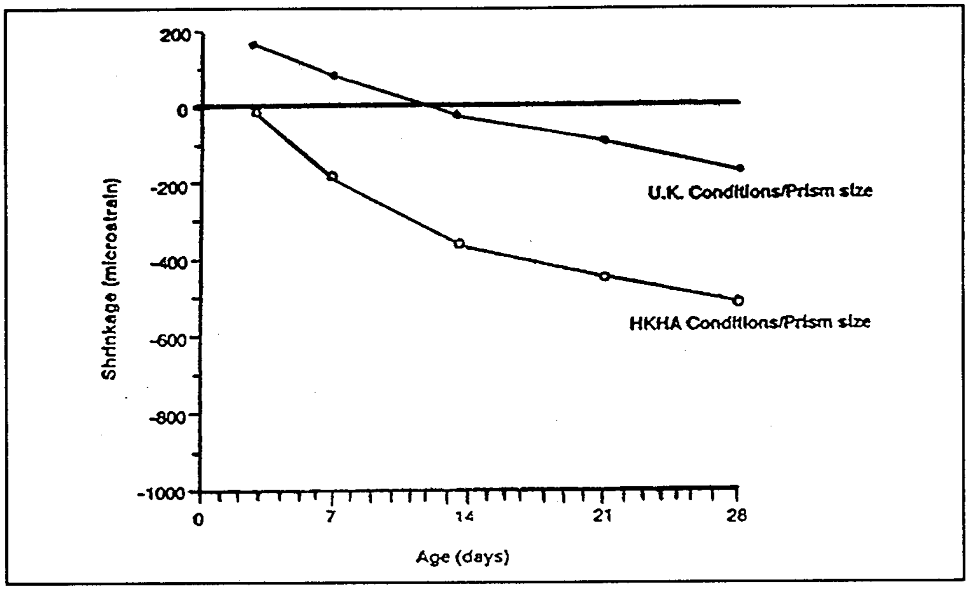


Figure 42. Mortar shrinkage results tested to HKHA and United Kingdom conditions (Decter and Lambe 1992)

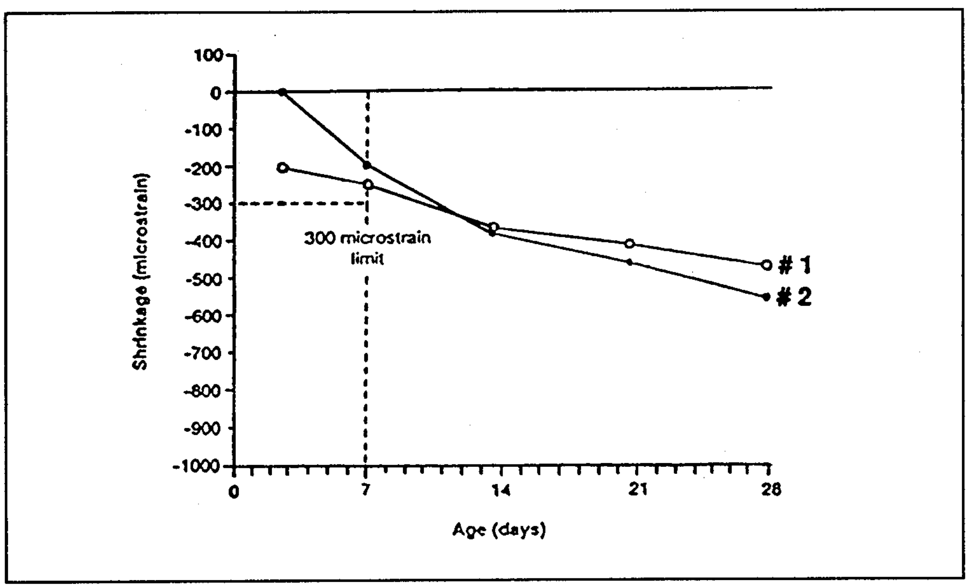


Figure 43. Plot of shrinkage of two mortars (Decter and Lambe 1992)

## Table 29
## Preliminary Performance Criteria

| Property | Test Method | Requirement | |
|---|---|---|---|
| Tensile strength, minimum | | | |
| 7 days | ASTM C 78 (ASTM 1994c) | 2.0 MPa | 290 psi |
| 28 days | ASTM C 496 (ASTM 1994j) | 2.75 MPa | 400 psi |
| Modulus of elasticity, maximum | ASTM C 469 (ASTM 1994h) | 25 GPa | $3.6 \times 10^6$ |
| Drying shrinkage, maximum at 28 days | ASTM C 157 (Modified) (ASTM 1994e) | 0.04% | |
| Cracking in 28 days | Cautinho Ring | 0 | |
| | German Angle | 0 | |
| | Volume Change Indicator (SPS Plate Test) | 0 | |
| Coefficient of thermal expansion, maximum | CRD-C 39-81 | $11.7 \times 10^{-6}/°C$ | $6.5 \times 10^{-6}/°F$ |

**Note:** CRD-C 39-81 (U.S. Army Engineer Waterways Experiment Station 1949) is listed in References section following main text.

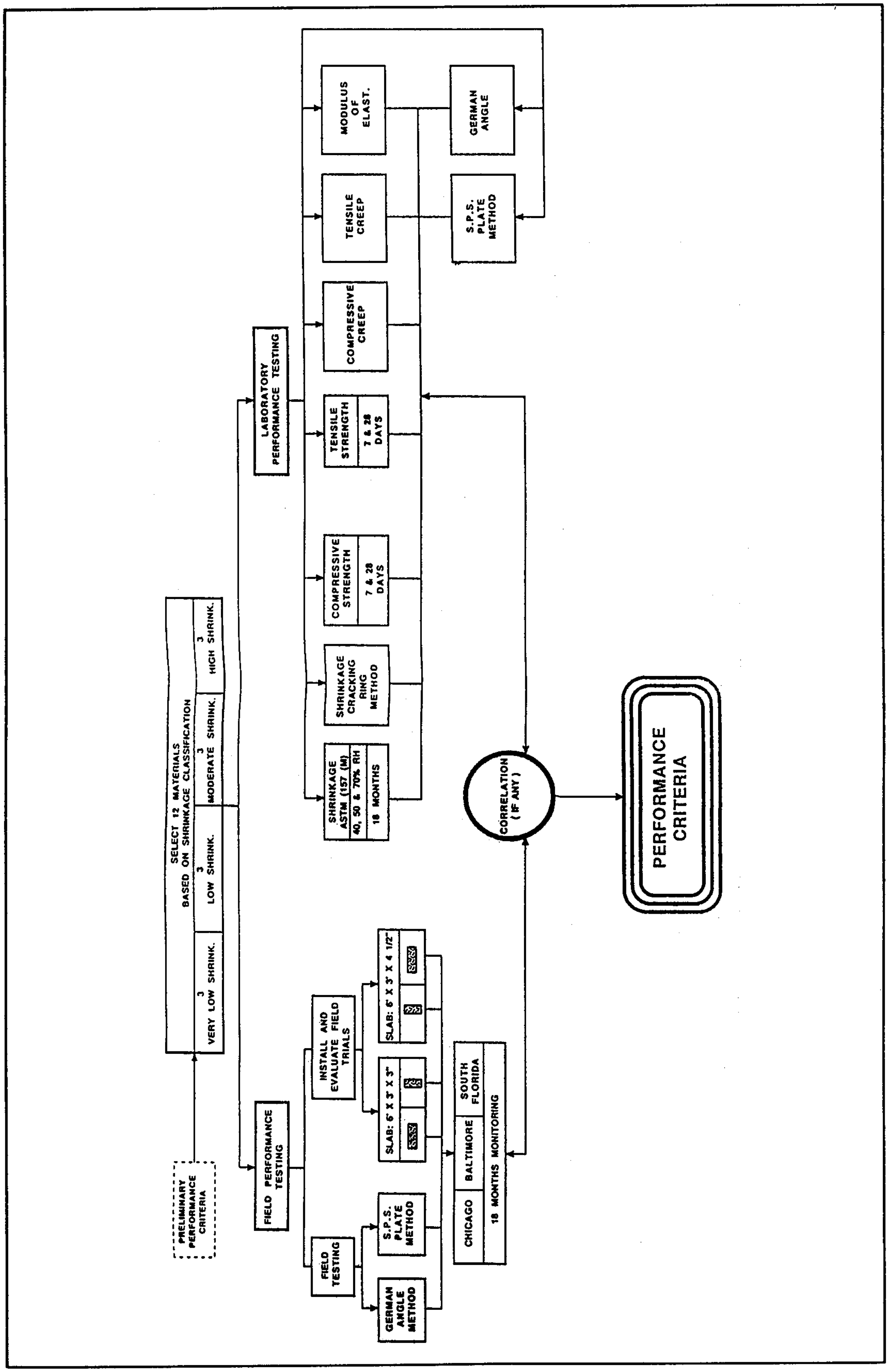


Figure 44. Performance criteria for repair materials. Phase II Experimental Program

## Conclusions and Research Needs

### Conclusions

The designer and prospective user of repair materials are not equipped with the rational analytical tool for selection of appropriate materials for a particular repair in a specified environment. This can be achieved by developing performance criteria for the selection of repair materials. As a first step in developing a performance criteria, preliminary performance criteria for dimensionally compatible repair materials are developed in accordance with the scope of this project. The preliminary criteria will be evaluated in laboratory and in situ conditions (Phase II).

Preliminary performance criteria specify the use of laboratory cast specimens only. Shrinkage criteria are expressed in terms of maximum drying shrinkage level at 28 days using the ASTM C 157 (Modified) test method. The criteria are intended for screening and selecting cement-based materials for concrete surface repairs. The criteria are preliminary for the following reasons:

*a.* The criteria are based on very limited data on field and laboratory shrinkage tests and performance.

*b.* The criteria should be verified further by being correlated with the field performance of concrete surface repairs subjected to various service conditions.

*c.* The criteria need to be assessed with regard to repeatability within and reproducibility within and among laboratories.

*d.* The effects of the placement conditions, curing conditions, and curing duration on the criteria need to be evaluated.

*e.* Field performance data are needed to determine if higher shrinkage values would provide satisfactory performance.

*f.* A notable limitation of the ASTM C 157 test is its relatively small sample size and the absence of restraint. Also, the limitation of imprecision exists. This test method modified by Alberta Transportation and Utilities is still considered to be the best available performance shrinkage test method for which performance data exist. Consideration in an experimental testing program is given to obtaining field performance and laboratory data and making correlations between them for other test methods described in this report.

Before meaningful performance criteria can be written, it will be necessary to establish a correlation between laboratory test results and actual field performance. The correlation study will be a part of large-scale laboratory and field tests and would include representatives of a wide range of concrete repair materials.

Large-scale field tests will have to be conducted in conjunction with the investigation of material performance issues discussed in this report. The specific issues addressed in this work are discussed in the experimental program.

### Research needs for developing performance criteria - experimental program

Because of the very limited data on which the preliminary criteria are based, the criteria should be evaluated on a trial basis. The criteria need to be assessed with regard to repeatability and reproducibility of laboratory results using statistically designed experiments and testing.

Field performance data are needed to evaluate the effects of different service conditions.

Most likely the preliminary criteria will need to be modified as additional field performance results and laboratory data and experience are obtained using the proposed test methods.

Of key importance is the selection of a reliable industry wide shrinkage test method and a determination of the shrinkage level that will result in satisfactory performance of repaired structures over the required period of time.

Based on the results of the proposed experimental program (Figure 44), performance criteria based on the selected shrinkage test method or a combination of the test methods will be developed.

# 6 Experimental Program for Phase II

The objective of Phase II of this study is to evaluate, under field and laboratory conditions, the preliminary performance criteria, shrinkage test methods, and to establish a performance criteria for the selection of repair materials based on their dimensional compatibility with existing concrete.

Twelve repair materials will be selected for laboratory and field evaluation based on the proposed shrinkage classification:

*a.* 1 for very low shrinkage (0.025 percent and less)

*b.* 4 for low shrinkage (0.026 to 0.05 percent)

*c.* 4 for moderate shrinkage (0.06 to 0.10 percent)

*d.* 3 for high shrinkage (higher than 0.10 percent)

## Laboratory Testing

Laboratory testing of the 12 repair materials will consist of standard ASTM test methods to determine the basic physical characteristics of each of the products:

*a.* Compressive strength at 7 and 28 days

*b.* Tensile strength at 7 and 28 days

*c.* Modulus of elasticity

*d.* Compressive creep (loaded to 20 percent and 40 percent of the ultimate compressive strength)

*e.* Coefficient of thermal expansion

In addition, a method will be selected or developed to test the tensile creep of the materials. The test procedures will also include the following shrinkage testing:

*a.* Shrinkage testing in accordance with ASTM C 157 (Modified) (ASTM 1994e) under relative humidities of 20, 50, and 90 percent with shrinkage monitored during an 18-month period.

*b.* Restrained shrinkage cracking ring method under laboratory conditions.

*c.* German Angle Method (German Federal Ministry for Transport, Highway Construction Department 1990) under laboratory conditions.

*d.* Restrained Volume Change Strain/Stress Indicator (SPS Plate method) under laboratory conditions.

The laboratory phase of the experimental program will also include the following studies:

*a.* Effect of prism size on drying shrinkage value tested by ASTM C 157 (Modified) using 3- by 3- by 11-in. and 1- by 1- by 11-in. concrete prisms.

*b.* Effect of polymer modification on drying shrinkage at 28, 60, and 90 days.

A number of observations will be made on the behavior, handling characteristics, and plastic properties of the tested products during the fabrication of the samples.

## Field Performance Testing

Three job sites are recommended within the scope of Phase II: Chicago, Arizona, and South Florida. Consideration in the selection of these locations was given to the following variables:

*a.* Temperature

*b.* Humidity

*c.* Wet-dry cycles

*d.* Freeze-thaw cycles

The 12 materials selected are to be installed at each site. The major variables to be investigated in the field is climate, with temperature and humidity being the most important climatic factors.

The following field testing is proposed for each material:

*a.* German Angle Method.

*b.* Restrained Volume Change Strain/Stress Indicator (SPS Plate method).

*c.* Installation of 6-ft by 18- by 3-in. sections and repairs in prefabricated concrete slabs (Figure 45).

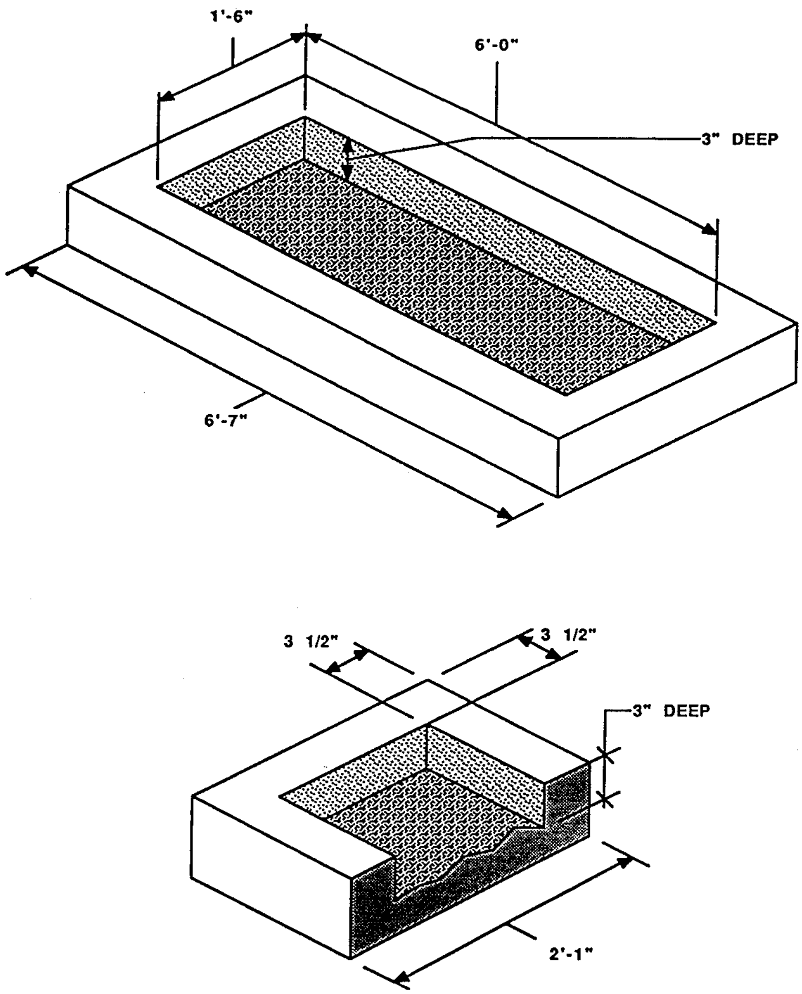


Figure 45. Proposed shrinkage test slabs

In the field application tests, 6-ft by 18- by 3-in. sections of each of the 12 systems will be placed. The purpose of this test will be to evaluate placement of these repairs under actual field conditions and to compare the performance of the repairs placed in field with the specimens fabricated and tested in the field and laboratory.

The experimental repairs are proposed for measuring the cracking which occurred due to the drying shrinkage. Crack lengths and average crack width will be measured and recorded for each repair slab. For each crack, the width is multiplied by the crack length to get the crack area. The sum of crack areas of all the cracks in the slab is called "total crack value" for the test repairs. The acceptable "total crack value" will be established after the experimental study is finalized. The attempt to establish a relationship between "total crack value," behavior of field samples, laboratory shrinkage, and other property test results will be made. The bond between the repair and the concrete substrate will be tested by the direct pull-off test method.

Field material mixing, placement, and testing at all sites should be closely observed and monitored by the Phase II Principal Investigator or a qualified representative. Although it is intended that materials be placed under actual field conditions, it is important that field applications result in uniform fabricated systems at all sites.

As far as possible, all systems at a given site should be placed successively, with a minimum time interval between each placement. This is because it is necessary to perform all tests at a site under essentially the same temperature and humidity conditions.

A program of field monitoring tests will be conducted on the repair materials as the primary basis for their evaluation. It is proposed that such monitoring be conducted during an 18-month period following the installation.

In addition to the foregoing monitoring, a series of photographs will be taken of each material (application) during each test round to serve as a record for later reference and report.

Attempts will be made to develop a suitable correlation between the material sample tests and field performance tests. These field tests will be the final stage in qualifying the repair materials for repair systems based on their dimensional compatibility and will provide the necessary information for developing performance criteria that addresses all of the relevant properties and performance variables for durable repair systems.

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a. ASTM C 31, "Standard practice for making and curing concrete test specimens in the field."

b. ASTM C 39, "Standard test method for compressive strength of cylindrical concrete specimens."

c. ASTM C 78, "Standard test method for flexural strength of concrete (using simple beam with third-point loading)."

d. ASTM C 109, "Standard test method for compressive strength of hydraulic cement mortars (using 2-in. or 50-mm cube specimens)."

e. ASTM C 157, "Standard test method for length change of hardened hydraulic-cement mortar and concrete."

f. ASTM C 172, "Standard method of sampling freshly mixed concrete."

g. ASTM C 190, "Standard test method for tensile strength of hydraulic cement mortars."

h. ASTM C 469, "Standard test method for static modulus of electricity and Poisson's ratio of concrete in compression."

i. ASTM C 490, "Standard practice for the determination of length change of hardened cement paste, mortar, and concrete."

j. ASTM C 496, "Standard test method for splitting tensile strength of cylindrical concrete specimens."

k. ASTM C 512, "Standard test method for creep of concrete in compression."

l. ASTM C 531, "Standard test method for linear shrinkage and coefficient of thermal expansion of chemical-resistant mortars, grouts, and monoliths surfacings."

m. ASTM C 596, "Standard test method for drying shrinkage of mortar containing portland cement."

n. ASTM C 666, "Standard test method for resistance of concrete to rapid freezing and thawing."

o. ASTM C 779, "Standard test method for abrasion resistance of horizontal concrete surfaces."

p. ASTM C 806, "Standard test method for restrained expansion of expansive cement mortar."

q. ASTM C 827, "Standard test method for change in height at early ages of cylindrical specimens from cementitious mixtures."

r. ASTM C 878, "Standard test method for restrained expansion of shrinkage compensating concrete."

s. ASTM C 882, "Standard test method for bond strength of epoxy-resin systems used with concrete by slant sheer."

t ASTM C 884, "Standard test method for thermal compatibility between concrete and an epoxy-resin overlay."

u. ASTM C 928, "Standard specification for packaged, dry, rapid-hardening cementitious materials for concrete repairs."

v. ASTM C 932, "Specification for surface-applied bonding agents for exterior plastering."

w. ASTM C 1090, "Standard test method for measuring changes in height of cylindrical specimens from hydraulic cement grout."

x. ASTM C 1107, "Standard specification for packaged dry, hydraulic-cement grout (nonshrink).

y. ASTM C 1202, "Standard test method for electrical indication of concrete's ability to resist chloride in penetration."

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The durability of concrete repairs depends to a large extent on the proper selection and application of repair materials. Restrained contraction of repair materials, the restraint being provided through bond to the existing concrete substrate, is a major factor which significantly increases the complexity of repair projects as compared to new construction. Preliminary performance criteria for dimensionally compatible repair materials were developed based on a state-of-the-art review of the literature on cement-based repair materials. These preliminary criteria are intended for screening and selecting cement-based materials for concrete surface repairs. A comprehensive experimental program of laboratory and field tests was developed to verify the performance criteria.

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