DIAGNOSTIC EVALUATION AND REPAIR OF DETERIORATED CONCRETE BRIDGES

FINAL REPORT

By

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Conducted by the

DEPARTMENT OF CIVIL ENGINEERING

UNIVERSITY OF MISSISSIPPI

In Cooperation with

THE MISSISSIPPI DEPARTMENT OF TRANSPORTATION

And

U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

The University of Mississippi

University, Mississippi

December 2004

Technical Report Documentation Page

1.Report No. FHWA/MS-DOT-RD-04-169	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Diagnostic Evaluation and Repair of Deteriorated Concrete Bridges		5. Report Date December 2004 6. Performing Organization Code
7. Author(s) Ahmed Al-Ostaz		8. Performing Organization Report No. MS-DOT-RD-04-169
9. Performing Organization Name and Address University of Mississippi Department of Civil Engineering University, MS 38677		10. Work Unit No. (TRAIS) 11. Contract or Grant No.
12. Sponsoring Agency Name and Address Mississippi Department of Transportation Research Division PO Box 1850 Jackson, MS 39215-1850		13. Type Report and Period Covered Final Report - April 2003 – November 2004 14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

Concrete bridge deck degradation has been observed in several bridges located in north Mississippi. In this study a survey was conducted to collect and document instances of deterioration in concrete bridges located throughout the State. The main causes of deterioration were observed to be corrosion, efflorescence, scaling and pop-outs. A guideline for identifying the basic causes of deterioration in the State of Mississippi and a guideline for the selection of suitable repair materials and methods were proposed. Preliminary work was conducted to evaluate the bond integrity of repair materials and concrete for a range of resin based materials and cement based materials which are used most commonly by MDOT maintenance staff.

17. Key Words Repair materials, repair techniques, concrete deterioration, bridge rehabilitation, thermal compatibility, concrete diagnostic.		18. Distribution Statement Unclassified	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages	22. Price

Form DOT F 1700.7 **(8-72)**

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ACKNOWLEDGMENT

This report include the results of a study titled “Diagnostic Evaluation and Repair of Deteriorated Concrete Bridges”, conducted by the Department of Civil Engineering, The University of Mississippi, in cooperation with the Mississippi Department of Transportation (MDOT). Funding of this project by MDOT is gratefully acknowledged.

The author wishes to thank Mr. Bill Barstis with MDOT Research Division for his efforts in coordinating the overall plan of the project. The help of MDOT Engineers and Assistant Engineers: Paul W. Swindoll, Jimmy Q. Dickerson, Walter G. Lyons, William R. May, Ricky Lee, Bennie Holmes, and Todd Jordan are gratefully acknowledged. Without the help of MDOT maintenance engineers and bridge inspectors this project would have not been completed. This includes: John Vance, Kenneth O’Neal Peach Jr., Lucy Hurst, Mitch Brazeal, James Magee, Donald W. MacDraw, Don Barrett, and Terry Sanders.

The author would like also to acknowledge the help of repair materials suppliers: James Hadley, Sika Corporation; Glenn Robinson, Polymer Concrete Inc.; Jim Watts, Hunt Process Inc.; Jack Shoemaker, Pot Fill Inc.; Tom O’Heron, Chemrex Inc.; and Brad Currier, Five Stars Company.

Sarat Ponnapalli, Brian Tenkhoff, Ryan Williams and Ryan Holmes were the key personnel from the University of Mississippi conducting laboratory work and providing support in the field. Also, the support of Gene walker in maintaining lab equipment is truly appreciated.

ABSTRACT

Concrete bridge deck degradation has been observed in several bridges located in north Mississippi. In this study a survey was conducted to collect and document instances of deterioration in concrete bridges located throughout the State. The main causes of deterioration were observed to be corrosion, efflorescence, scaling and pop-outs. A guideline for identifying the basic causes of deterioration in the State of Mississippi and a guideline for the selection of suitable repair materials and methods were proposed. Preliminary work was conducted to evaluate the bond integrity of repair materials and concrete for a range of resin based materials and cement based materials which are used most commonly by MDOT maintenance staff.

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CHAPTER 1

LITERATURE REVIEW

The US economy is supported by a vast network of transportation infrastructure facilities in the form of highways, railroads, waterways, transit ways, pipelines etc. The worth of this infrastructure is estimated to be around $2.5 trillion. In the US there are about four million miles of paved roads and highways and 575,000 bridges. Statistics from the Federal Highway Administration (FHWA) indicated that approximately 230,000 bridges are either functionally obsolete or structurally deficient and are in need for immediate rehabilitation requiring a total investment of $70 billion [1]. In another study [2], (FHWA) estimated that: nearly half of the bridge inventory is deficient due to either structural or traffic inadequacies; a $90 billion backlog of bridge maintenance exists; traffic congestion wastes 1.4 billion gallons of gas and 1.2 billion person-hours each year; and transportation delays add $7.6 billion annually to costs in the US [3]. It is estimated that $78B will be spent over the next 20 years in major rehabilitation of bridges [4]. However, this expenditure is only able to maintain the *status quo*, i.e., as many bridges become newly deficient as are refurbished [5]. More than a third of the highways are in poor or mediocre condition. Increased traffic and larger trucks place greater loads on highways and bridges. By 2005, it is estimated that inadequate roads will cost the economy $50B per year [6]

Consequently, the economic well-being of the nation, the safety of citizens, and the quality of life are all being adversely impacted. The US Federal Reserve Board has concluded that the failure of civil infrastructure systems to perform at the expected level might reduce the national gross domestic product (GDP) by as much as 1%. Studies by the National Bureau of Standards (NBS, now the National Institute for Standards and Technology) estimated that overall corrosion costs in the United States are 4.2% of the Gross National Product (GNP) or over $300 billion [7].

Deterioration of concrete structures is a safety issue in addition to being an economic issue. Undetected or unheeded corrosion of bridges and other structures can cause catastrophic failure with loss of life. Two of the most well known corrosion-induced bridge collapses are the Point Pleasant (Silver) Bridge over the Ohio River in 1967 and the Mianus River Bridge on I-95 in Connecticut in 1983 [8] The Silver Bridge failed from corrosion cracks in an eye-bar while corrosion of a pin-and-hanger assembly caused the Mianus River Bridge collapse. Forty-seven people died during the Point Pleasant Bridge collapse. The cost in 1967 was $175M; the cost of the same disaster today is estimated to be $2.1-5.6B [5]. Other bridges have required emergency or accelerated repairs, closure, or traffic restrictions as a result of extensive corrosion, including [8]:

- Harvard Bridge in Cambridge, MA
- Yankee Doodle Bridge (I-95) in Norwalk, CT
- Southeastern Pennsylvania Transportation Authority (SEPTA) Bridge in Philadelphia
- Williamsburg Bridge in New York City
- Ben Franklin Bridge in Philadelphia
- Royal Gorge Bridge in Colorado
- Portsmouth Bridge over the Ohio River
- Tower Bridge in London
- Lake Maracaibo Bridge in Venezuela.

This report shows that the State of Mississippi has its share of the problem as well. Problems of corrosion, sulfate attack, efflorescence and scaling are major threats to the well being of the State transportation system.

The deteriorated state of the nation’s infrastructure [9] has led the Civil Engineering Research Foundation to recommend the use of alternative materials that have attributes of lower cost, lighter weight, and reduced maintenance and enhance durability [8]. Polymeric materials have seen a significant utilization increase in structures such as bridges, decks, columns, roads, pipes and high-rise buildings.

Prior to commencing repair of concrete structures, it is always advisable to investigate the possible causes of this deterioration [10]. Nothing will be gained by carrying out a repair if the deterioration is likely to commence immediately. Causes of deterioration may be divided into recurring and non-recurring. If the recurrence of deterioration is acceptably low, then it is normally acceptable to restore the structure as nearly as possible to its original state. If, however, there is an unacceptable risk of recurrence, the structure should be repaired and the fundamental cause of deterioration should be controlled to acceptable limits [11]. Once the cause of deterioration is known, a decision of the extent of repair is required. This includes the parameters of durability, strength, function, and appearance of the structure after the repair process is completed. After the above decision is made, the choice of repair material and repair technique could be investigated.

Literature review has been carried out on the following aspects:

- Concrete Deterioration and Symptoms
- Diagnostic Technique
- Extent of Deterioration and Repair
- Feasibility of the repair

1.1 Concrete Deteriorations and Symptoms

Concrete structures are inherently durable and usually require a minimum of repair and maintenance. However, there are occasions when damage in defects requires remedial treatment to be carried out. Before carrying out any remedial measures on a concrete structure, it is most important to identify the basic causes which have made repair necessary. Otherwise, an inappropriate and consequently ineffective repair technique may be selected. This Current Practice Sheet is a general introduction to assist the reader in identifying likely causes of defects or deterioration. It deals both with the defects apparent shortly after construction and also with deterioration which occurs after many years of use. However, aspects specific to roads, floor topping and renderings are not included.

In some cases, e.g. fire damage, the cause is obvious. In other cases, the causes may only be established by means of detailed programs of testing and examination. Generally, the first step is to visually inspect the structure and review its history.

In most cases, deterioration of a concrete structure can be attributed to one or more of the following causes with the symptoms included with each cause:

1. Structural deficiency

This may result from errors in either design or construction or alternatively from improper or altered use of a structure. It is usually characterized by cracking in highly stressed regions and the cracking is commonly perpendicular to the main reinforcement. Foundation movement can also be considered a structural deficiency; however, the resulting cracks are not necessarily perpendicular to the main reinforcement [10].

2. Corrosion of reinforcement

Steel embedded in concrete does not normally corrode. Nevertheless, when there is insufficient cover, areas of poor compaction, or large amounts of chloride present, rusting may occur and because rust occupies considerably more volume than steel, stresses are set up which cause cracking or spalling of the overlying concrete. Characteristic of this type of damage is cracking or spalling which follows the line of the reinforcement, and in fact emanates from it [11].

3. Chemical attack

Chemical attack occurs when aggressive liquids of damp chemicals are in contact with concrete. Etching or softening of the surface may result (e.g. acidic attack). Alternatively, the concrete may crack and spall due to sulfate attack [12].

4. Fire damage

Fire damage can result in cracking and spalling of concrete. There is usually no difficulty in identifying the cause and methods are available to evaluate the extent of the deterioration.

5. Internal reaction in the concrete

In certain rare circumstances, reactions can occur between cement and substances present in the aggregates, resulting in expansive forces with subsequent cracking and spalling. For example, map or 3-legged cracking associated with an exuding gel can be symptomatic of alkali-aggregate reaction. Swelling with cracking and spalling can be a result of sulfates present in the aggregate. These reactions are slow and damage is not normally noticed until many years after construction [13].

6. Restrained thermal contraction and expansion

Concrete, in common with other materials, contracts and expands with variations of temperature and when the contraction or expansion is restrained, damage can result. Temperature variations can occur either as a result of externally applied heat or cold or due to the quite considerable quantities of heat generated by concrete during setting and hardening.

In a large volume of fresh concrete, the heat generated is not easily dissipated and a substantial rise in temperature can result. If precautions are not taken to limit the difference in temperature between the interior and the surface of the concrete, the temperature differential can result in cracking.

In other situations with smaller volumes, the heat generated during setting and initial hardening raises the temperature of the concrete and when the hardened concrete subsequently cools and attempts to contract, cracking may occur if restraint is present. Common examples of such cracking are: the parallel, regularly spaced cracks which often occur when concrete is cast against older concrete which has already hardened, the cracks which occur in walls or panels bridging between fixed parts of a structure.

Heat from external sources produces thermal expansion and if this is restrained, large local compressive stresses can be set up and result in spalling. Thermal contraction and expansion sometimes cause pre-existing cracks to open and close with variation in ambient temperature.

7. Restrained shrinkage

As concrete initially sets and hardens, it shrinks slightly; subsequent, additional shrinkage occurs as it dries out. If this shrinkage is restrained, cracking may result. Restrained initial shrinkage resembles in many ways the restrained thermal contraction and the situations in which damage occurs and the type of damage are similar. Usually, such damage is a combination of the two effects. Cracks due to restrained shrinkage are often noticed soon after construction, but in cases of slow drying as a major factor, they may not be apparent till much later [14].

8. Creep

Inadequate design which fails to allow for creep of the structured elements of a building (e.g. shortening of columns or deflection of floors and beams) may result in the load being transferred to non-structural elements such as partition walls or cladding panels, where cracking and damage will often result. Creep is a long term effect and its consequences are only apparent after a period of years.

9. Rapid early evaporation

Rapid loss of water from concrete at an early age, frequently leads to plastic shrinkage cracking in the form of alligator like cracks. This phenomenon usually occurs (but may not always be noticed) within two hours of casting and is particularly prevalent in slabs cast in situations exposed to wind or sun [15].

10. Plastic settlement

Another form of cracking present within hours of casting is plastic settlement cracking which occurs when fresh concrete hangs up on either reinforcement or formwork. It typically occurs in columns, deep beams, or walls with mixes which have a tendency to bleed.

11. Miscellaneous inadequate construction

Other faults which can require repair include: insufficient cover to steel, honeycombing or voids in the concrete, blemishes such as conspicuous or excessive blow-holes which are not structurally significant but are aesthetically displeasing.

Application of these signs to cases of deterioration in Mississippi will be presented in Chapter 3.

1.2 Diagnostic Techniques

A repair program cannot succeed unless a correct diagnostic evaluation has been carried out [16-21]. Diagnostic evaluation includes identification of the basic causes which have caused deterioration and made repair necessary, evaluation of the extent of damage, and feasibility of repair.

1.2.1 Identification of the causes:

i) *Visual inspection*: This leads to a technical interpretation of the symptoms and modes of deterioration [19, 20, and 22].

ii) *Collection of data pertaining to the structure*: This includes review of documents and interviews to obtain information on specifications, drawings, soil reports, reports on construction and inspection measures, and reports related to quality control on concrete constituent materials and concrete mixes [19, 20, 22, and 23].

iii) *Detailed program of testing and examination*: This requires obtaining samples for laboratory investigations and employing a range of non-destructive testing techniques to obtain relevant information. Information will be needed on chloride content, sulfate content, alkalinity (ph) of hardened cement paste specially in the proximity of concrete-steel interface, depth of carbonation, concrete permeability, density, strength, resistively, pulse velocity characteristics, loss of metal in reinforcement, cover to reinforcement, active/passive state of reinforcement, crack widths and shape, aggregate quality, composition of the mix, rate of progress of deterioration, and active/passive state of deterioration [22,24-27].

1.2.2 Review of Inspection Methods and Diagnostic Techniques

An inspection survey [22, 28-31] of a deteriorated structure is an essential prerequisite for preliminary diagnosis and an evaluation of the condition of the structure. Input from this general survey facilitates decisions on repair feasibility and repair options. Several technical papers [19, 20, and 32] describe and illustrate typical symptoms of commonly occurring forms of concrete deterioration with an objective to assist in the diagnosis of the causes of defects or deterioration. Such a global survey has to be followed up with a range of relevant site and laboratory tests to plan repair strategies. These include chloride determination in concrete [33, 34], depth of concrete cover [28, 35-38], mix analysis and half cell potential measurements [39, 40]. Some of these tests can be carried out non-destructively on site; others can be carried out either partially destructive or distinctive in the laboratory. Commonly used non-destructive or partly destructive in-situ tests are ultrasonic pulse velocity measurements (50-52) , depth of cover measurements with pachometer, rebound Schmidt hammer observations [44,45], pull-off, break-off, penetration, internal practice and surface hardness tests. Amongst the more commonly used of these tests are the pulse velocity method [41-43], rebound Schmidt hammer [44, 45] and Windsor Probe [46, 47]. More detailed descriptions of these tests will be discussed later in Table 1.3.

Tests on cores [48-51] constitute a semi-destructive technique but provide valuable information on strength, porosity, density, carbonation, electrical resistively, moisture content, chloride content, mix proportions, water absorption, pulse velocity, gamma radiography and an assessment of damage due to sulfate attack and other chemical reactions.

1.2.3. Inspection Methods and Testing Techniques used for Diagnosis of Deterioration

A. Guidelines for inspection Surveys and Identifying Deterioration Causes Through Visual Symptoms

ACI 201. 1R-68 [22] provides a checklist for making a survey of the condition of concrete. It also defines and illustrates typical forms of deterioration which afflict concrete. Idorn [29] in his monograph on “durability of concrete Structures in Denmark” has also described a format for making survey inspections of deteriorated structures with the objective to identify the nature and extent of deterioration. Axon [30, 31] and Olson [31] have also described possible formats for carrying out condition surveys of concrete in service. Several technical papers [19, 28, and 32] have appeared which describe and illustrate typical symptoms of commonly occurring forms of concrete deterioration with an objective to assist in the diagnosis of the causes of defects or deterioration. These include cracking due to plastic shrinkage, plastic settlement, drying and restrained shrinkage, restrained thermal contraction and expansion, alkali-aggregate reaction and crazing; deterioration due to sulfate and other chemical attack, freezing and thawing, water cavitations, fire, creep and corrosion of reinforcement; defects such as honeycombing, scaling, stains, sand streaking, pop outs and spalls. ACI Committee 224 [52] has reported comprehensively on "Causes, Evaluation and Repair of Cracks in Concrete Structures." This report includes cracking in plastic and hardened concrete due to environmental factors, thermal stresses, chemical reactions, weathering, corrosion of reinforcement, poor construction practices, overloading and errors in design and detailing.

B. Site and Laboratory Testing Techniques for Concrete Evaluation

1. <u>Chloride Determination in Hardened Concrete</u>

Chloride induced corrosion of reinforcement with associated cement cracking and spalling is one of the important forms of concrete deterioration in Mississippi. Therefore, the relationship between chloride concentration in the concrete and severity of corrosion of steel reinforcement is considered the most relevant parameter in evaluating the corrosion damage potential of a reinforced concrete structure. The determination of the chloride content of cement is therefore one of the most important and directly relevant laboratory/site tests in the process of diagnosis and potential damage evaluation. The chloride content of the concrete is most likely to significantly affect the repair selection and implementation procedures.

The Volhard laboratory method is fully described in BS 1881 pt. 6 [33] and Building Research Establishment has described procedures in their two information sheets [53,54]. “Hach” simplified method makes use of a commercially available kit [55], and the “Quantab” simplified method also uses a commercially available Quantab strip [56] to measure the chloride concentration of solution made from powdered concrete. Berman [57], Browne and Bolling [58] and Clear and Harrigan [59] have described detailed chemical procedures for evaluating the extent of chloride contamination of concrete. In addition to the chemical tests, x-ray fluorescence spectrometry techniques can also be used to determine the chloride content of the cement. Petersen and Paulsen have described a rapid chloride test using a chloride testing kit [60]. In terms of latest developments, a 24-State pooled fund study in the USA has produced equipment for determining the chloride content of concrete at the level of the reinforcing steel non-destructively [61]. The equipment uses two nuclear procedures, prompt capture gamma-ray analysis and neutron activation analysis to make the chloride measurement. The instrument has been tested on five Texas bridges and chloride results were found to be quite satisfactory in comparison with the wet chemistry methods.

2. *Measurement of Alkalinity of Concrete (pH)*

The alkalinity of the hardened cement paste is normally in excess of 12 and very often in excess of 12.5 due to the presence of $Ca(OH)_2$ and alkalis. Mild steel reinforcement is passivated in concrete against corrosion when pH is greater than 10. When pH is depressed below 10 due to carbonation and/or chloride attack, the vulnerability of steel to corrosion increases; below a pH of 8 steel is clearly open to corrosion. Several investigators [62,63] have found pH measurements on or near actively corroding rebar sites (anodes) to be in the range of 4.5 to 6. pH measurements, therefore, provide an excellent indication of the vulnerability of the steel to corrosion in the concrete environment.

A simple, although approximate, pH determination technique has been described by Rasheeduzzafar et al [64]. FHWA has developed a procedure [65] for measuring the pH at small sites along the fracture faces of hardened concrete. The method employed is patterned after the work of Hartt at Florida-Atlantic University and includes a flat tipped pH electrode, indicator solutions and indicator papers. The technique can be used to measure the pH at anode (corroding) and cathode (non-corroding) sites along fracture faces of concrete. Precise and most reliable estimation of concrete alkalinity can be made by determining the pH of the pore fluid extracted from hardened concrete specimens after a year of hydration using a specially designed pressure vessel. The equipment and the method have been fully described by Longuet et al [66], Barney and Diamond [67] and Page and Vennesland [68].

3. *Electrical Resistivity of Concrete*

Corrosion of reinforcement embedded in concrete is an electrochemical process [35,62,69]. Thus, the magnitude of the corrosion current is primarily controlled by the resistivity of concrete. High resistivity reduces current and also the probability of corrosion. The electrical resistivity of concrete ranges from around 103 ohm-cm, when saturated, to 1011 ohm-cm, when oven dried. For normal moist concrete, the value is around 104 ohm-cm. It has been shown by Rasheeduzzafar et al [36] that corrosion may increase seven fold when the concrete resistivity value decreases from 15000 ohm-cm to 6000 ohm-cm. Several investigators [70, 71] have described techniques to measure concrete resistivity. Rasheeduzzzafar et al [64] have described a technique using an integrated compact instrument that eliminates a frequently occurring source of error due to spurious potentials or stray currents.

4. *Depth of Concrete Cover*

This parameter appears to be the single most important design and construction practice parameter affecting the durability performance of concrete against corrosion of reinforcement in this region. Depth of cover measurement is not only a most important measurement in its own right but is also of great significance for interpreting chloride contents and carbonation depth in the concrete. The procedure for measuring depths of concrete cover is well known and easily employed on site. It is completely non-destructive and requires only a knowledge of the rebar diameter. Rasheeduzzafar et al [36,37] have described a technique for measuring cover depth using a commercial Covermeter. The Covermeter is effective for depths of cover up to about 3 inches (80mm). If the Covermeter measurements are carried out on a grid system over the concrete surface, equi-depth cover contours can be constructed. Which clearly illustrate the variability in depth of cover and in any regions where it is less than satisfactory.

5. *Measurement of the Degree of Carbonation*

During drying, the pore water in the concrete evaporates and is replaced by air which contains carbon dioxide and other acidic gases which react with the alkaline constituents of the concrete thereby reducing the degree of the concrete alkalinity. The normal protection against corrosion provided by the concrete is lost as a result of carbonation and corrosion of steel reinforcement will occur if moisture and oxygen are available. Evaluation techniques for carbonation of concrete have been studied by number of investigators [34, 72]. W/C ratio, cement composition, age of concrete, and exposure conditions are the primary controlling factors. On site, the simplest and the best technique of measuring the depth of carbonation is by exposing a fresh concrete surface using a hammer and chisel and then spraying this surface with a 2 percent solution of phenolphthanlein ethanol which is a pH indicator. Magenta color is usually observed in the un-carbonated concrete. The color change occurs at a pH of about 10.

6. *Assessment of Concrete Strength and Quality*

An estimation of concrete strength and its general quality is essential for evaluating current and future structural safety and the progressive deterioration potential of a structure. A direct measure of the in-situ cube or cylinder strength cannot be conveniently obtained simply because it is not easily possible to produce a cast cubic or cylindrical specimen from that location. However, it is possible to obtain an estimate of in-situ cube/cylinder strength of concrete by using core tests or other in-situ or non-destructive testing (NDT) methods including ultra sonic pulse velocity, pull-out, break-off, penetration, internal fracture and surface hardness tests. Among the more commonly used of these methods are core tests, the pulse velocity method, rebound Schmidt hammer, and Windsor probe.

Tests based on concrete cores constitute a semi-destructive technique. Only a limited number of cores can be taken in any one location followed by repairs. However, a large number of tests can be performed very reliably on extracted cores; these include strength, porosity, permeability, density, carbonation, resistively, moisture content, chloride analysis, mix proportions, water absorption, pulse velocity, gamma radiography and an assessment of damage due to sulfate attack and other chemical reactions. Several investigators [48-51] have discussed the technique comprehensively in terms of its application to the evaluation of concrete strength, general quality and the development of other data useful and directly relevant to the concrete durability and deterioration problems.

Pulse velocity method is a non-destructive technique and has come into increasing use to provide the background information needed to plan more intensive investigation in selected areas. The relatively low cost, the speed with which results can be obtained, the ability to evaluate the whole construction generally, and the timely nondestructive nature of the technique make it a most valuable technique in the exploration of the general quality of concrete and an indirect measurement of the strength. Leslie and Cheesman [41], White Hurst [42] and Jones [43] have discussed the technique comprehensively and have provided a basis for evaluating concrete condition. Calibration correlation curves for a particular concrete can provide an excellent means of evaluating strength quantitatively. Pulse velocity techniques can be used to detect cracks, honeycombing and sometimes the extent of cracking and honeycombing. Commercially available compact equipment are readily available for use.

7. *Rebound Schmidt hammer*

Rebound Schmidt hammer is another useful tool for examining the condition of structures [44, 73]. The technique is used for measuring surface strength or hardness which has been empirically related to bulk concrete strength. The Schmidt rebound hammer consists of a plunger held in contact with the concrete surface and a spring loaded mass strikes the free end and rebounds. The extent is an approximate guide to the strength and usually shows a wide scatter but the instrument is simple and easy to use.

Windsor Probe is the third tool which has been frequently used to measure in-situ concrete strength in a range of structures [46, 47]. The device is easy to use and rarely requires surface preparation prior to testing. Basically, the test consists of shooting a standard probe into the concrete with a standard cartridge. The extent of penetration is measured and this is related empirically to concrete strength. The method is used for locating areas of weak concrete but it has the disadvantage that it is partly destructive.

For all the three above NDT techniques (pulse velocity, Schmidt hammer and Windsor probe), as Arni [74] has pointed out, correlation has to be established between the NDT methods and the strength of concrete in the individual construction preferably on the basis of core strengths.

8. *X-rays and Gamma rays*

X-rays [73] and Gamma-rays [75] are used to investigate rebar location and density of concrete. In gamma radiography, the reflected intensity of backscatter of the rays is related to the density of the concrete and can, therefore, be used to locate voided and poorly compacted concrete. The equipment is portable and comparatively easy to use and long exposure times are not needed as in the case of X-ray photography. The backscatter method has the disadvantage that it effectively only examines the concrete within 50mm of the surface. Tomographic techniques [76] for isolated specimens have revealed a variety of laws and features but the extension to structural surveys needs much development. The method normally relies on through transmission which would not always be possible for in-situ applications. Scattering or reflection of these radiations is potentially dangerous due to the high intensities generated for penetration. One example where gamma radiography has been used with some success on site, however, was for a survey on a 1920's built concrete road bridge [77]. The survey, which was conducted within carefully controlled safety criteria, provided detailed information related to the reinforcement, its sizes and location. It also enabled the engineers to make judgments on the integrity of the bridge.

The X-ray technique provides an internal picture of the concrete by placing the X-ray source on one side of the concrete member and a sensitive plate on the other. The method shows up voids in the concrete, but is not often used due to the high cost and extensive exposure times required.

9. *Concrete Permeability and Water Absorption Measurements*

Permeability of concrete is the pre-eminent criterion governing the durability performance of concrete in aggressive environments. Concrete permeability determination in the field has met with little success and even in the controlled conditions of the laboratory; it is a difficult test to perform. Several low pressure and high pressure permeability tests have been developed and described in literature [78-81]. Tyler and Erlin [82] have described a high pressure permeability test. Rasheeduzzafar et al [83] have also developed a high pressure permeability test in connection with concrete durability testing and research. Water absorption measurements being known to be intimately connected with the permeability characteristics, a 30-minute absorption test and an initial surface absorption test developed by Levitt [83] are the frequently used water absorption tests. Both these tests are fully described in BS 1881 Part 5. A test has also been developed by Figg [84] which measures the properties of the surface skin (or cover) concrete in a relatively non-destructive manner. Figg test has been modified by

Richards [85], Building Research Establishment [86], Partners [87] and Kasai [88]. Whiting [89,90] has also developed a chloride permeability test for concrete.

10. <u>Half-ell potential measurements</u>

Effectively this method measures the electrode potential of steel reinforcing bars in the concrete environment by comparison with the known electrode potential of a reference electrode (half cell) which by definition must maintain a constant value. It has been known for a number of years that the electrode potential of steel in concrete is an indicator of corrosion activity. In reinforced concrete, the electrode potential determines the feasibility of the electrochemical reaction which results in the corrosion of the steel reinforcement.

The reference electrode generally used in measurements on reinforced concrete is the saturated copper/copper sulfate electrode and the values of electrode potential of reinforcement, *E*, generally accepted as representing corroding and non-corroding conditions are as follows:

- *E* > -0.20 volt is an indication of less than 5% probability of corrosion.
- -0.20 volt > *E*> -0.35 volt is an indication of probability of corrosion is uncertain.

The surface of concrete to be investigated is normally divided up into a grid system of suitable dimensions. The reinforcement in a structure is usually all in good electrical contact so that only one electrical connection to the reinforcement is needed, but if there is a doubt over electrical continuity, additional connections can be made or continuity tested using a test meter. The potential difference between the reinforcement and the half cell is measured using a high impedance voltmeter. The two most commonly employed half cells are the saturated calomel electrode and the saturated copper/coppers sulfate electrode. The latter is more durable and hence better suited to site work. It can easily be made from readily available materials and will maintain a steady potential over extended periods.

A summary of evaluation procedure versus desirable properties of concrete is shown in tables 1.1-1.3 below.

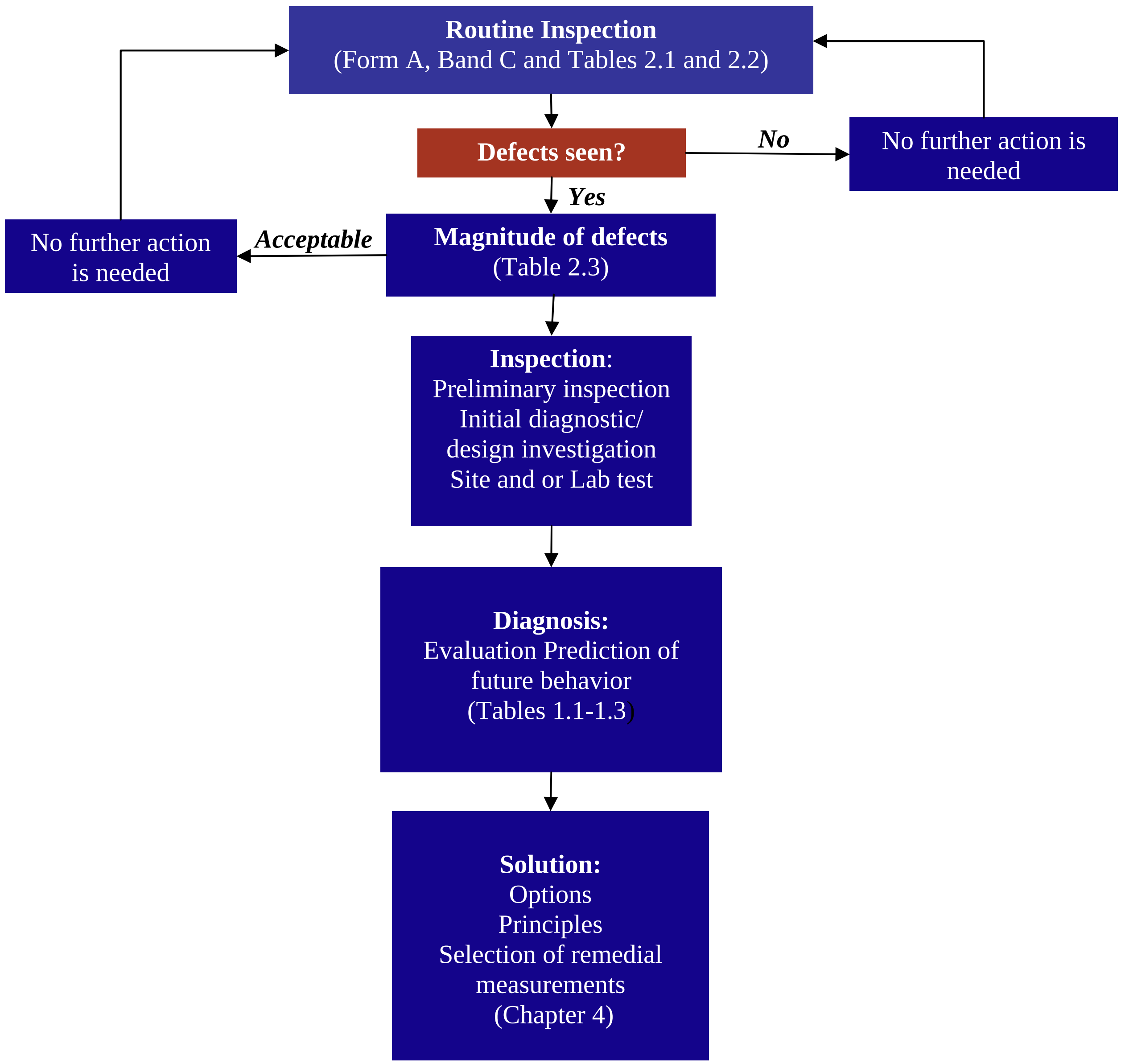

Table 1.1(a) – Evaluation of concrete properties [129]

Evaluation Procedure / Chemical And Physical Properties	Acoustic Impact (Table 1.3)	Air Content test (ASTM C457)	Cement Content Test (ASTM C1084)	Chemical Tests	Core Testing	Electrical potential measurements (Table1.3)	Electrical resistance measurements (Table1.3)	Flexural tests(ASTM C42)	Freeze thaw test (ASTM C666)	Gamma radiography (Table 1.3)	Nuclear moisture meter (Table 1.3)	Permeability test (CRD C48)	Petrographic analysis (ASTM C856)	Pullout testing(ASTM C900)	Rebound hammer (ASTM C805)	Ultrasonic pulse (ASTM C597)	Windsor probe (ASTM C803)
Acidity				●									●				
Air content		●											●				
Alkali-Carbonate Reaction													●				
Alkali-Silica Reaction													●				
Cement content			●	●									●				
Chemical Composition				●									●				
Chloride content				●	●								●				
Compressive strength					●									●	●	●	●
Contaminated Aggregate				●									●				
Contaminated Mixing water				●									●				
Corrosion environment				●		●											
Creep					●												
Density					●					●							
Elongation					●												
Frozen components													●				
Modulus of elasticity					●											●	
Modulus of rupture					●			●									
Moisture Content					●		●				●						
Permeability												●	●				
Pullout strength														●			
Quality of aggregate													●				
Resistance to Freezing and Thawing					●				●				●				
Soundness					●					●			●				

Splitting tensile strength					●												
Sulfate resistance				●									●				
Tensile strength					●			●									
Uniformity	●												●		●		●
Water cement ratio													●				

Table 1.1(b) – Evaluation of physical conditions of concrete [129]

Evaluation Procedure / Physical condition	Acoustic Emissions (Table 1.3)	Acoustic impact (Table 1.3)	Chemical Tests	Core Testing(ASTM C42)	Fiber optics (Table 1.3)	Gamma Radiography (Table 1.3)	Infrared thermography (Table 1.3)	Load testing (ACI 437R)	Petrographic analysis (ASTM C856)	Physical measurement	Radar (Table 6.3)	Rebound hammer (ASTM C805)	Ultrasonic pulse (ASTM C597)	Ultrasonic pulse echo (Table 1.3)	Visual examination(ACI 201.1R, ASTM C823)	Windsor Probe (ASTM C803)
Bleeding channels									●						●	
Chemical Deterioration			●						●						●	
Corrosion of steel			●	●					●						●	
Cracking	●	●		●	●		●		●	●	●		●	●	●	
Cross-sect. Properties and thickness				●		●				●			●			
Delamination		●		●	●	●	●		●		●		●	●	●	
Discoloration			●						●						●	
Disintegration				●		●	●		●				●		●	
Distortion															●	
Efflorescence			●						●						●	
Erosion									●						●	
Freeze-Thaw damage									●						●	
Honeycomb				●	●	●	●		●				●		●	
Popouts															●	
Scaling															●	
Spalling				●		●	●								●	
Stratification		●			●									●	●	
Structural performance	●							●							●	
Uniformity of concrete					●				●			●	●		●	●

Table 1.2- Evaluation of properties of reinforcing steel [129]

	Acoustic impact(table 1.3)	Chemical analysis(ASTM A571)	Coating tests(ASTM A775,G12,14,20)	Cover meters pachometer(table 1.3)	Electrical potential measurements(table 1.3)	Gamma radiography(table 1.3)	Physical measurements	Radar(table 1.3)	Tension tests(table 1.3)	Ultrasonic pulse echo(table 1.3)	Visual inspection
Adhesion of epoxy coating			●								
Anchorage							●				
Bend test							●				
Breaking strength									●		
Carbon content		●									
Chemical composition		●	●								
Coating Properties		●									
Concrete cover				●		●	●	●			
Continuity of epoxy coating			●								
Corrosion					●		●				●
Cross sectional properties and thickness							●				
Deformations							●				●
Elongation									●		
Exposure											●
Rebar location	●			●		●	●	●			
Reduction of area									●		
Shape							●				
Strength of connections							●				
Tensile strength									●		
Thickness of epoxy coating			●								
Weld shear strength							●				
Yield strength									●		

Table 1.3-Description of nondestructive (event as noted) evaluation methods for concrete [129].

Method	Applications	Principle of operation	User expertise	Advantages	Limitations
Acoustic emission (Clifton et al.1982)	Continuous monitoring of structure during service life to detect impending failure; monitoring performance of structure during proof testing	During crack growth or plastic deformation, the rapid release of strain energy produces acoustic(sound) waves that can be detected by sensors in contact with or attached to the surface of a test object	Extensive knowledge required to plan test and interpret results	Monitors structural response to applied load; capable of locating chance of possible failure; equipment is portable and easy to operate, good for load tests	Expensive test to run; can be used only when structure is loaded and when flaws are growing; interpretation of results required an expert; currently largely confined to laboratory; limited track record, further work required.
Acoustic impact (Clifton et al.1982	Used to detect debonds, delaminations, voids and hair line cracks	Surface of object is struck with an implement. The frequency and damping characteristics of resulting sound giving an indication of the presence of defects; equipment may vary from simple hammer or drag chain to sophisticated trailer mounted electronic equipment	Low level of expertise required to use auditory system but the electronic system requires training	Portable equipment; easy to perform with auditory system; electronic device requires more equipment	Geometry and mass of test object influence result; poor discrimination for auditory system; reference standards required for electronic testing
Core testing (ASTM c42)	Direct Determination of concrete strength; concrete evaluation of condition type and quality of aggregate, cement and other components	Drilled cylindrical core is removed from the structure; tests may be performed on core to determine compressive and tensile strength, torsional properties, static modulus of elasticity etc.	Special care not to damage cores must be taken in obtaining drilled cores; moderate level of expertise required to test and evaluate results	Most widely accepted method to determine reliably the strength and quality of in-place concrete. Good for examination of cracks embedded reinforcing bars and for sample for chemical tests	Coring damages structures and repairs may be required. Destructive test.
Cover meters/Pach-ometers (Malhotra 1976)	Measure cover size and location of reinforcement and metal embedments in concrete or masonry	Presence of steel in concrete or masonry affects the magnetic field of a probe. The closer the probe e is to steel the greater the effect	moderate; easy to operate training needed to interpret results	Portable equipment, good results if concrete is lightly reinforced. Good for locating reinforcing or prestressing tendons and wires to avoid damage in coring	Difficult to interpret results if concrete is heavily reinforced or if wire mesh is present. Not reliable for cover of 4in. And form ties often mistaken for anchors.
Electrical Potential Measureme-nts (Mathey and Clifton 1988)	Indicating condition of steel reinforcing bars in concrete masonry. Indicating the corrosion activity in concrete pavements	Electrical potential of concrete indicates probability of corrosion	Moderate level of experience required, user must be able to recognize problems	Portable equipment, field measurements readily made; appears to give reliable information	Information on rate of corrosion is not provided; access to reinforcing bars required
Electrical resisitance measureme-nts (Mathey and Clifton 1988)	Determination of moisture content of concrete	Determination of moisture content of concrete is based on the principle that the conductivity of concrete changes with changes in moisture content	High level of expertise required to interpret results; equipment is easy to use	Equipment is automated and easy to use	Equipment is expensive and requires high frequency specialized applications; dielectric properties also depend on salt content and temperature of specimen, which poses problems in interpretation of results. Not too reliable.
Fiber optics(Mathey and Clifton 1988)	To view the portions of a structure that are inaccessible to the eye	Fiber optic probe consisting of flexible optical fibers, lens and illuminating system is inserted into a crack or drilled hole in concrete; eyepiece is used to view interior to look for flaws such as cracks voids or aggregate debonds; commonly used to look into areas where cores have been removed or bore holes have been drilled. Examination of cavity walls and other masonry holes	Equipment is easy to handle and operate	Gives clear high resolution images of remote objects. Camera attachment for photos is available. Flexible hose enables multi directional viewing	Equipment expensive; many bore holes are required to give adequate access. Mortar in masonry walls hinders view

Infrared thermograp-hy(Mathey and Clifton 1988)	Detection of internal flaws, crack growth, delamination, and internal voids	Flaws detected by using selective infrared frequencies to detect various passive heat patterns which can be identified as belonging to certain defects. Through cracks in concrete and masonry may be detected on cold days.	High level of expertise required to interpret results	Has potential for becoming a relatively inexpensive and accurate method for detecting concrete defects; can cover large areas quickly	Requires special skill and equipments. Effective where temperature differential between surfaces is high
Load testing(ACI 437R)	Determine performance of a structure under simulation of actual loading conditions, using overload factors	Test load is applied to structure in a manner that will simulate the load pattern under design conditions	High level of expertise required to formulate and conduct the test program and to evaluate the results. Protection shoring is required for safety	Provides highly reliable prediction of structure's ability to perform satisfactorily under expected loading conditions	Expensive and time consuming; testing may cause limited or even permanent damage to the structure or some of its elements
Nuclear moisture meter(ASTM D 3017)	Estimation of moisture of hardened concrete	Moisture content in concrete is determined based on the principle that materials(such as water) decrease the speed of fast neutrons in accordance with amount of hydrogen produced in test specimen	Must be operated by trained and licensed personnel	Portable moisture estimates can be made of in-place concrete	Equipment sophisticated and expensive; NRC license required to operate; moisture gradients in specimen may give erroneous results. Measures all nitrogen in concrete as well as nitrogen in water
Petrographic analysis(ASTM C 856)	Used to determine a variety of properties of concrete or mortar sample removed from structure; some of these include 1)dense ness of cement 2)homogeneity of concrete3) location of cracks4)air content5)proportions of cement, aggregate and air voids6)curing	Used in conjunction with other tests chemical and physical analysis of concrete samples is performed by qualified petrographer	High level of skill training required to perform and analyze test results	Provides detailed and reliable information of concrete ingredients, paste, aggregates, curing, possible damage, and freezing.	Qualified experienced petrographer required; relatively expensive and time consuming
Pullout testing(ASTM C 900)	Estimation of compressive and tensile strengths of existing concrete	Measure the force required to pullout the steel rod with enlarged head cast in concrete; pullout forces produce tensile and shear stresses in concrete	Low level of expertise required, can be used by field personnel	Directly measures in-place strength of concrete; appears to give good prediction of concrete strength	Pullout devices must be inserted during construction; cone of concrete may be pulled out, necessitating minor repairs.
Pull-off testing(Long and Murray 1984)	Estimation of the compressive strength of existing concrete	Circular steel probe is bonded to concrete. Tensile force is applied using portable mechanical system until concrete fails. Compressive strength can be estimated using calibration charts	Highly skilled operator is not required	Simple and inexpensive	Standard test procedure not yet available. Limited tack record. Concrete must be repaired at test locations
Radar(Mathey and Clifton 1988)	Detection of substratum voids, delaminations, and embedments. Measurement of thickness of concrete pavements	Uses transmitted electro magnetic impulse signals for void detection	High level of expertise required to operate equipment and interpret results	Expedient methods can locate reinforcing bars and voids regardless of depths. May be used when only one surface is available	Equipment is expensive; reliability of void detention greatly reduced if reinforcement present; procedure still under development
Gamma radiography(Malhotra 1976)	Estimation location, size and condition of reinforcing bars; voice in concrete; density	Based on principle that the rate of absorption of gamma rays is affected by density and thickness of test specimen; gamma rays are emitted from source, penetrate the specimen, exit on opposite and are recorded on file	Use of gamma producing isotopes is closely controlled by NRC, equipments must be operated by licensed inspectors	Internal defects can be detected; applicable to variety of materials; permanent record on film; gamma ray equipment easily portable	Equipment is expensive; gamma ray source is health and safety hazard; requires access to both sides of specimen

Rebound hammer(ASTM C 805)	Compares quality of concrete from different areas of specimen; estimates of concrete strength based on calibration curves with limited accuracy	Spring driven mass strikes surface of concrete and rebound distance is given in R values; surface hardness is measured and strength is estimated from calibration curves provided by hammer manufacturer	simple to operate; can be readily operated by field personnel	Equipment is light weight, simple to operate and inexpensive, large amount of data can be quickly obtained; good for determining uniformity of concrete and stress potentially low strength	Results affected by condition of concrete surface; does not give precise prediction of strength; estimates of strength should be used with great care; frequent calibration of equipment is required.
Ultrasonic pulse(ASTM C 597)	Gives estimates of uniformity quality compressive strength(when previously correlated) of concrete; internal discontinuities can be located and their size estimated; most widely used stress wave method for field use	Operates on the principle that stress wave propagation velocity is affected by quality of concrete; pulse waves are induced in materials and the time of arrival measured at the receiving surface with a receiver.	Varying level of expertise required to interpret results. Operator requires a fair degree of training	Equipment relatively inexpensive and easy to operate; accurate measurement of uniformity and quality. By correlating compressive strength of cores and wave velocity, in-situ strength can be estimated	Good coupling between transducer and concrete is critical; interpretation of results can be difficult; density, amount of aggregate, moisture variations and presence of metal reinforcement may affect results; calibration standards required
Visual Examination(ACI 201.1R and ASTM C 823)	(a)Evaluation of the surface condition of concrete (finish, roughness, scratches, cracks, color) (b) Determining deficiencies in joints c) Determining deformations and differential movements of structure.	Visual examination with or without optical aids, measurement tools, photographic records, or other low cost tools; differential movement determined over long periods with surveying methods and other instrumentation.	Experience required to determine what to look for, what measurements to take, interpretation of conditions, and what follow up testing to specify.	Generally low costs; rapid evaluation of concrete conditions.	Trained evaluation required primary evaluation confined to surface of structure.
Penetration resistance(ASTM C 803)	Estimates of compressive strength, uniformity and quality of concrete may be used for estimating strength prior to form removal.	Probes are gun driven into concrete; depth of penetration converted to estimates of concrete strength by using calibration curves.	Simple to operate, can be readily operated in the field with little training. Safety requires operator certificate.	Equipment is simple, durable, and requires little maintenance, useful in assessing the quality and relative strength of concrete; does relatively little damage to specimen.	May not yield accurate estimates of concrete strengths; interpretation of results depends on correlation curves. Difficulty in removing the probes, which are often broken and damaging to cover concrete.
Ultrasonic pulse echo(Thornton and Alexander 1987)	Gives estimates of compressive strength, uniformity and quality of concrete. Can locate reinforcing bar, defects, voids delamination and determine thickness.	Operates on principle that original direction, amplitude, and frequency content of stress waves introduced into concrete are modified by presence of interfaces such as cracks, objects, and sections which have different acoustic impedance.	High level of expertise required to interpret results. Operator should have considerable training to use equipment and knowledge of electronics, and should have considerable training in the area of condition survey of concrete structures.	Can operate where only one surface is accessible. Can operate in dry(in theory- never saw publicized material). Allows one to “see” inside concrete.	Is still in development stage. Needs development of measurement criteria. Not presently a standard test method. Digital signal processing can improve interpretation but data must be returned to laboratory for processing at present.
Resonant frequency testing(Carino and Sansalone 1990)	Is used in the laboratory to determine various fundamental modes of vibration for calculating modulii; used in field to detect voids, delaminations.	A resonant frequency condition is set up between two reflecting interfaces. Energy can be introduced by hammer impact oscillator-amplifier-electromagnetic driver system.	High level of expertise required to interpret results. Operator can be easily trained for laboratory measurements as specimens have simple geometry.	Allows one to “see inside” concrete structures, can penetrate to depths of a number of feet; a newly developed transducer receiver can improve results over an accelerometer.	Operates in sonic range and does not have resolution of ultrasonic. Still in developing stage.

1.3 *Evaluation of the extent of damage*

The extent of damage is evaluated on the basis of three criteria:

i. Extent of deterioration in relation to acceptable/unacceptable levels of severity so established as to commensurate with the local conditions.

ii. Extent of deterioration in relation to the degree of negation of efficiency related to functional/serviceability aspects.

iii. Extent of deterioration in relation to structural reserves and actual or imminent risks of structural failure.

The evaluation of the extent of damage would involve an estimation on the basis of inspection surveys, collection of data, investigative testing and strength calculations, the following aspects of deterioration for the various cases are:

i. Average loss of metal and extent of pitting in corrosion afflicted reinforcement.

ii. Loss of strength of corroded reinforcement due to degradation of metal and reduction in steel area.

iii. Loss of bond between concrete and reinforcement as a result of formation of corrosion products and concrete spalling and delamination.

iv. Loss of strength due to insufficient cover, honeycombed and porous low quality concrete.

v. Reduction in the flexural stiffness due to cracking in the tension zone resulting in additional deflections and bond slip due to overloading.

vi. Reduction in concrete section due to sulfate attack.

vii. Reduction in concrete strength due to sulfate attack.

viii. Width of cracks.

1.4 Feasibility of Repair

This phase of diagnostic evaluation relates to a decision, on the basis of information on deterioration causes, mechanisms and the extent of damage, whether a repair is possible and economically viable. No literature is currently available on this aspect of repair work.

CHAPTER 2

GUIDELINE FOR VISUAL INSPECTION, TECHNIQUES EMPLOYED, DURABILITY SURVEYS AND INSPECTION OF SURVEY RESULTS

2.1 Guideline for Visual Inspection of Bridges in Mississippi

Common types of deterioration symptoms and construction faults in concrete bridges are: spalling and delamination of concrete due to corrosion expansion which leaves reinforcement exposed; loss of metal of the reinforcement due to oxidation of steel during the corrosion process; loss of strength of corroded reinforcement due to degradation of metal and reduction in steel area; loss of bond between concrete and reinforcement due to the formation of corrosion products and concrete spalling; reduction in the flexural stiffness due to cracking in the tension zone resulting in additional deflection and bond slip due to overloading; cracking due to various causes; reduction in the strength and cohesion of concrete due to sulfate attack; initial construction defects such as insufficient cover to reinforcement, and excessively honeycombed and porous concrete of low quality.

Three documents have been developed for carrying out visual inspections of concrete deterioration in existing structures as follows:

A- Visual inspection of concrete deterioration in existing structures- Background.

B- Visual inspection of concrete deterioration in existing structures- Observation of deteriorations.

C- Visual inspection of concrete deterioration in existing structures- Simplified defect classification.

Overall inspection procedure is summarized in the chart below:

Form A: Visual inspection of concrete deterioration in existing structures- background information

---

Identification of structure:

District:_________________________ File: ____

City: ___________________________ Year: ____

Locality:_________________________ Date: ____

Structure:____

Construction: _____________________ Inspector: ___________________________________

Age of Structure: ____

Mass concrete ☐ Reinforced concrete ☐ Pre-stressed concrete ☐

Topography of site:

Mountain ________________________ Salina _______________________________________

Lowland _________________________ Swamp ______________________________________

Coast ___________________________ Others ______________________________________

City ☐ Urban ☐ Rural ☐

Exposure Conditions

- ☐ Tide
- ☐ Submersion
- ☐ Sea Water
- ☐ Constant wind
- ☐ Constant wind with dust
- ☐ Traffic/Wear
- ☐ Apparent Overloading
- ☐ Ground Water

Max day temperature (summer) ____

Max night temperature (summer) ____

Max day temperature (winter) ____

Max night temperature (winter) ____

Maximum humidity ____

Others: ____

Concrete Data:

Cement:____

Sand: ____

Aggregates

- ☐ Light Weight Chert
- ☐ Dense Chert
- ☐ Small Maximum Size Chert
- ☐ Lime Stone (Kentucky)
- ☐ Lie Stone (Alabama)

Water:

- ☐ Tap (drinking)
- ☐ Surface
- ☐ Well
- ☐ Sea

Admixtures

- ☐ Pozzolans
  - Fly Ash
  - Slag
- ☐ Plasticizers ____
- ☐ Air-entrainment ____

Initial concrete quality: Poor ☐ Good ☐ Excellent ☐

Supplementary information:

____

____

____

____

Date: ______________________

Signature: ____________________

Form B: Visual inspection of concrete deterioration in existing structures- observations of Deterioration.

Symptoms of deterioration	Extent of damage and photograph			
	Light	Medium	Heavy	Severe
Crazing	1	2	3	4
Map-cracking				
Random cracking				
Coarse, single crack				
Plastic settlement cracks				
Plastic shrinkage cracks				
Drying shrinkage cracks				
Thermal cracks				
Shear cracks				
Reinforcement corrosion cracks				
Encrustations				
Stalactites				
Resinous gel				
Flakes of dry gel				
Efflorescence				
Surface scaling				
Physical salt weathering				
Sulfate attack expansion and cracking				
Sulfate attack - loss of strength into cohesionless				
Granular mass				
Floor heave				
Pitting				
Pop-outs				
Erosion				
Disintegration				
Rust staining				
Reinforcement corrosion				

Spalling .. ..

Uncovered/exposed reinforcement .. .. ..

General quality Condition ____

Repairs None ☐ Partial☐ Overall☐

Method of repairs ____

Supplementary documentation

Type File No.

Sketches __________

Photographs __________

Samples:

Concrete __________

Exudations __________

Aggregates __________

Remarks: ____

Detailed inspection (Table 2.1) Required ☐ Not required☐

Date:_________________________

Signature:_____________________

Table 2.1 Checklist for detailed inspection [12]

Decks

- Are there any cracks? Do they leak? What is the location, direction, width, and depth?
- Is the surface sound, or are there areas where surface scaling is present?
- Is any steel reinforcement exposed?
- Is there any evidence of concrete Delamination?
- Is there any evidence of corrosion of reinforcing steel or surface spalling?
- Are there any signs of leakage? Describe conditions and location.
- If there is a traffic-bearing membrane are there any tears, cracks, or loss of adhesion?
- Are there low spots where ponding occurs?
- Are there water stains on the underside of the deck?

Beams and Columns

- Are there any cracks? If so, what is the location, direction, width, and depth?
- Are there any signs of leakage? Describe conditions and note location.
- Is there any concrete spalling?
- Is any steel reinforcement exposed?
- Are bearings in good condition?
- Are bearing plates rusted?
- If bearing pads have been used under beams, are they present and in good condition? Are bearing pads squashed, bulging, out of place, or missing?

Isolation joints and expansion joints

- Are there any leaks through isolation-joint seals and expansion-joint seals?
- Are leaks related to failure of seals or adjacent concrete?
- Could the cause be snowplows?
- What type of isolation joint or expansion joint seal is installed?
- Who is the manufacturer?
- Is there a warranty in force?
- Consult the manufacturer for repair recommendation if applicable.

Joint sealants

- Are there any signs of leakage, loss of elastic properties, separation from adjacent substrates, or cohesive failure of the sealant?
- If bearing pads have been used under beams, are they present and in good condition? Are bearing pads squashed, bulging, out of place, or missing?

Exposed steel

- Is there any exposed steel (structural beams, handrails, door frames, barrier cable, exposed structural connections)?

- Is there any exposed embedded reinforcing steel or connections due to the spalling or chipping of concrete cover?
- Is rust visible?
- Is it surface rust or is there significant loss of section?
- Is repainting required?
- What is the condition of attachment point and surrounding concrete?

Drains

- Are the drains functioning property? When were they last cleaned?
- Are the drains properly located so that they receive the runoff as intended?
- Is the seal around the drain base in good condition?

Previous repairs

- Are previous repairs performing satisfactorily?
- Are the edges of previous patches tight?
- Does the patch sound solid when tapped?

General Comments

- Are records of previous inspections available? Have they been reviewed?
- Are there previous engineering reports available? Have they been reviewed?
- Has the concrete been tested for chloride content? Are reports available? Have they been reviewed
- Other comments

Project:: ____
Inspected by: ____
Date: ____

Note: A copy of the inspection report should be added to the operations/maintenance manual each time an inspection is under-taken.

Form B: Visual inspection of concrete deterioration in existing structures- simplified defect classification

Code	Feature	Description	Cause	Details to be given on Inspection	Remarks
A1	Cracking (general)	Jagged separation of concrete from no gap upwards	Overload, corrosion, shrinkage	Direction, width and depth	Cracking
A2	Pattern cracking	As cracking but formed as pattern	Differential volume change between internal and external concrete	Surface area, width	
B1	Exudation	Viscous gel – like material exuding through a pore	Alkali aggregate reaction	Severity	Surface
B2	Incrustation	A crust (white) on the concrete surface	Leaching of lime from cement	Severity/dampness	
B3	Rust stains	Brown stains	Corrosion of rebar, tying wire or surface steelwork	Severity	
B4	Dampness	The extent of water on the surface should be stated	Leakage, rundown	Severity	
C1	Pop-out	Shallow, conical depression	Development of local internal pressure, i.e, expansion of aggregate particle	Surface area, depth	Concrete loss
C2	Spall	A fragment detached from a larger mass	Exertion of local internal pressure, i.e. by rebar corrosion or exertion of external force	Area, depth	
C3	Delamination	A sheet spall	Exertion of internal pressure over a large area	Area, depth	
C4	Weathering	Loss of the concrete surface	Environmental action wears away the laitance and paste	Area, depth	
D1	Tearing	Similar to cracks	Adhesion to slip form shuttering	Width, depth	Construction loss
D2	Honey combing	Voids between coarse aggregates	Lack of vibration (consolidation)	Area associated	
E1	Construction Joint	Line on concrete surface, may be feather-edged or porous-looking	Joint between two pores	Area associated deterioration	Construction features
E2	Panel joint	Ridge in the concrete surface	Mark formed by shutter joint	Any associated deterioration	

Table 2.2 Guidelines on techniques employed in durability surveys.

	Properties examined	**Testing techniques**	**Other considerations**
Mechanical ON SITE	Concrete surface Quality	Schmidt hammer rebound number- Type N equipment	Errors due to surface softening, cracks or laminations, etc
LABORATORY	Compressive / tensile strength	Test on Core	
Chemical ON SITE	Carbonation Chloride, sulfate and moisture contents	Depth measurement using phenolpthalin spray Drilling to collect dust samples in 10 to 25 mm bands through the concrete using percussive drill	 Analysis to find penetration rate coefficient
LABORATORY	Concrete mix proportions Cement type Original water / cement ratio Minerological composition	Analysis of hardened concrete Aluminum: Iron ratio C_3S:C_2S ratio X-ray diffraction	
Physical ON SITE	Condition of embedded steel Condition of embedded steel Condition of embedded steel Condition of Cover	Electrochemical potentials Electrical resistively measurement Corrosion mapping procedure Fe depth meter- cover, reinforcement size and spacing	Careful interpretation using ASTM C876-80 interrelated with experience and results from other NDT techniques. Based on soil resistance measurement Problems where cover over 100mm, where steel mesh is used
LABORATORY	Permeability of cover Coefficient of thermal expansion Concrete density / porosity General (steel and concrete)	Water permeability – oxygen diffusion coefficient Visual and microscopic examination using optical and polarized light microscopes	Very susceptible to concrete preconditioning, i.e. laboratory conditions of storage. Usually determined using permeability testing Identification of aggregate source, presence of slag, presence of FA, etc

Table 2.3 Guidelines for interpretation of inspection survey results.

1. Corrosion of the reinforcement

Technique	**Interpretation**	**Measurement**
	mV	**chance of corrosion**
Electrochemical Potentials (CSE)	less *–ve* than -200	5%
	-200 to 350	50%
	more *–ve* than -350	95%
Concrete Resistively	**Ω cm**	**corrosion rate**
	<5000	Very high
	5000-10000	High
	10000-20000	Low
	>20000	Negligible
Cover to reinforcement	1. Observance of specification 2. Likely forms of deterioration due to rebar corrosion 3. Assessment of time to activation (see also chloride below)	
Chloride drillings	1. Activation of the reinforcement possible with levels >0.35-0.4 wt% of the cement 2. Assessment of time to activation.	
Core samples through rebar	Assessment of type and amount of corrosion	

2. Quality of the concrete cover

Schmidt N-type Hammer	**Comparative hardness of the cover zone**.	
	Average rebound No.	***Quality***
	>40	Good, hard layer
	30-40	Fair
	20-30	Poor
	<20	Concrete delaminated near surface
Pundit (indirect method)	>4 km/sec	Good quality cover
	3-4 km/sec	Fair quality cover
	<3 km/sec	Poor quality cover
Concrete cores	***Assessment for:*** 1. Visual assessment of integrity 2. Concrete strength 3. Concrete permeability to liquid and gas 4. Diffusion to gas 5. Depth of carbonation 6. General chemical/physical test	

2.2 Evaluation of Extent of Deterioration in Mississippi

2.2.1 Causes of deterioration

The commonly occurring deteriorations, symptoms and construction faults in Mississippi are:

i) Cracking

ii) Initial construction defects such as no-cover to reinforcement, honeycombed and porous concrete of low quality

iii) Thermal shrinkage

iv) Spalling and delamination of concrete due to corrosion expansion leaving reinforcement exposed.

v) Traffic accidents

vi) Scaling and wear

vii) Efflorescence

viii) Pop-outs due to alkali-silica reactions

2.2.2 Analysis of major deterioration signs in Mississippi

A questionnaire was send to all districts in Mississippi to identify the main cause of deterioration of concrete structures. The results obtained are tabulated below. Analysis of these results (Figure 2.1-2.4) shows that at least 20 % of the bridges need repair and rehabilitation. Efflorescence, corrosion, scaling, pop-outs and construction defects are the main signs of deterioration in Mississippi.

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School of Engineering
Department of Civil Engineering
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Please take a moment to fill out the following survey regarding bridges and overpasses in your MDOT district. Your answers will help develop a statistical analysis of the deterioration problems facing bridges and overpasses throughout the state. The results will be used to help develop methods and suggestions for the repair of Mississippi bridges and overpasses. Your participation is greatly appreciated.

Mississippi Department of Transportation District **District 2**

Number of bridges/overpasses in excellent condition ___**300**___
no deterioration; only routine maintenance required

Number of bridges/overpasses in good condition ___**360**___
little deterioration including non-structural cracking and light wear; no corrosion observed

Number of bridges/overpasses in fair condition ___**240**___
deterioration including corrosion, cement paste hydrolysis/efflorescence, scaling, pop-outs, etc.; structural problems could arise due to deterioration

Number of bridges/overpasses in poor/failing condition ___**197**___
severe deterioration affecting the structural soundness of the bridge or damage caused by impact or overloading

Total number of bridges/overpasses in district ___**1097**___
includes 18 all-timber bridges and 235 box bridges

(continued on next page)

Please indicate the number of bridges in your district suffering each mechanism or manifestation of deterioration. A particular bridge may exhibit signs of more than one type of deterioration or none at all. Therefore, your total number in this section may not add to your total number of bridges.

Corrosion (Spalling, Delamination, Cracking, etc.)	**65**
Cement Paste Hydrolysis (Efflorescence)	**400***
Light/Medium Scaling	**32**
Heavy/Severe Scaling	**16**
Pop-outs (Alkali-Silica Reaction)	**30***
Sulfate Attack	**0**
Construction Defects (Honeycombs, etc.)	**40***
Overload Damage	**15**
Collision Damage	**100**
Severe Wear	**40***

James M. Magee, Bridge Inspector	March 5, 2004
Signature	Date

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Please take a moment to fill out the following survey regarding bridges and overpasses in your MDOT district. Your answers will help develop a statistical analysis of the deterioration problems facing bridges and overpasses throughout the state. The results will be used to help develop methods and suggestions for the repair of Mississippi bridges and overpasses. Your participation is greatly appreciated.

Mississippi Department of Transportation District **District 3**

Number of bridges/overpasses in excellent condition **241**
no deterioration; only routine maintenance required

Number of bridges/overpasses in good condition **25**
little deterioration including non-structural cracking and light wear; no corrosion observed

Number of bridges/overpasses in fair condition **3**
deterioration including corrosion, cement paste hydrolysis/efflorescence, scaling, pop-outs, etc.; structural problems could arise due to deterioration

Number of bridges/overpasses in poor/failing condition **0**
severe deterioration affecting the structural soundness of the bridge or damage caused by impact or overloading

Total number of bridges/overpasses in district **269**

(continued on next page)

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Please indicate the number of bridges in your district suffering each mechanism or manifestation of deterioration. A particular bridge may exhibit signs of more than one type of deterioration or none at all. Therefore, your total number in this section may not add to your total number of bridges.

Corrosion (Spalling, Delamination, Cracking, etc.)	**28**
Cement Paste Hydrolysis (Efflorescence)	**37**
Light/Medium Scaling	**125**
Heavy/Severe Scaling	**53**
Pop-outs (Alkali-Silica Reaction)	**42**
Sulfate Attack	**0**
Construction Defects (Honeycombs, etc.)	**21**
Overload Damage	**0**
Collision Damage	**72**
Severe Wear	**0**

Donald W. McDraw, Jr., Bridge Inspector March 17, 2004
Signature

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Please take a moment to fill out the following survey regarding bridges and overpasses in your MDOT district. Your answers will help develop a statistical analysis of the deterioration problems facing bridges and overpasses throughout the state. The results will be used to help develop methods and suggestions for the repair of Mississippi bridges and overpasses. Your participation is greatly appreciated.

Mississippi Department of Transportation District **District 5**

Number of bridges/overpasses in excellent condition ___________**747**__________
no deterioration; only routine maintenance required

Number of bridges/overpasses in good condition ___________**329**__________
little deterioration including non-structural cracking
and light wear; no corrosion observed

Number of bridges/overpasses in fair condition ____________**22**__________
deterioration including corrosion, cement paste
hydrolysis/efflorescence, scaling, pop-outs, etc.;
structural problems could arise due to deterioration

Number of bridges/overpasses in poor/failing condition ____________**17**__________
severe deterioration affecting the structural soundness of
the bridge or damage caused by impact or overloading

Total number of bridges/overpasses in district __________**1115**__________

(continued on next page)

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Please indicate the number of bridges in your district suffering each mechanism or manifestation of deterioration. A particular bridge may exhibit signs of more than one type of deterioration or none at all. Therefore, your total number in this section may not add to your total number of bridges.

Corrosion (Spalling, Delamination, Cracking, etc.)	**500**
Cement Paste Hydrolysis (Efflorescence)	**300**
Light/Medium Scaling	**150**
Heavy/Severe Scaling	**0**
Pop-outs (Alkali-Silica Reaction)	**150**
Sulfate Attack	**0**
Construction Defects (Honeycombs, etc.)	**10**
Overload Damage	**20**
Collision Damage	**15**
Severe Wear	**0**

Don Barrett, Bridge Inspector
Signature

February 19, 2004
Date

THE UNIVERSITY OF MISSISSIPPI

School of Engineering
Department of Civil Engineering
LEADERSHIP IN ENGINEERING EDUCATION

Please take a moment to fill out the following survey regarding bridges and overpasses in your MDOT district. Your answers will help develop a statistical analysis of the deterioration problems facing bridges and overpasses throughout the state. The results will be used to help develop methods and suggestions for the repair of Mississippi bridges and overpasses. Your participation is greatly appreciated.

Mississippi Department of Transportation District **District 6**

Number of bridges/overpasses in excellent condition **7**
no deterioration; only routine maintenance required

Number of bridges/overpasses in good condition **503**
little deterioration including non-structural cracking and light wear; no corrosion observed

Number of bridges/overpasses in fair condition **146**
deterioration including corrosion, cement paste hydrolysis/efflorescence, scaling, pop-outs, etc.; structural problems could arise due to deterioration

Number of bridges/overpasses in poor/failing condition **36**
severe deterioration affecting the structural soundness of the bridge or damage caused by impact or overloading

Total number of bridges/overpasses in district **690 exc. Box bridges**

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Please indicate the number of bridges in your district suffering each mechanism or manifestation of deterioration. A particular bridge may exhibit signs of more than one type of deterioration or none at all. Therefore, your total number in this section may not add to your total number of bridges.

Corrosion (Spalling, Delamination, Cracking, etc.)	**400**
Cement Paste Hydrolysis (Efflorescence)	**400**
Light/Medium Scaling	**150**
Heavy/Severe Scaling	**1**
Pop-outs (Alkali-Silica Reaction)	**300**
Sulfate Attack	**2 box bridges**
Construction Defects (Honeycombs, etc.)	**40**
Overload Damage	**40 deck cracking**
Collision Damage	**75**
Severe Wear	**1**

Terry Sanders, Bridge Inspector	April 8, 2004
Signature	Date

THE UNIVERSITY OF MISSISSIPPI
School of Engineering
Department of Civil Engineering
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Please take a moment to fill out the following survey regarding bridges and overpasses in your MDOT district. Your answers will help develop a statistical analysis of the deterioration problems facing bridges and overpasses throughout the state. The results will be used to help develop methods and suggestions for the repair of Mississippi bridges and overpasses. Your participation is greatly appreciated.

Mississippi Department of Transportation District **District 7**

Number of bridges/overpasses in excellent condition **309**
no deterioration; only routine maintenance required

Number of bridges/overpasses in good condition **234**
little deterioration including non-structural cracking and light wear; no corrosion observed

Number of bridges/overpasses in fair condition **66**
deterioration including corrosion, cement paste hydrolysis/efflorescence, scaling, pop-outs, etc.; structural problems could arise due to deterioration

Number of bridges/overpasses in poor/failing condition **14**
severe deterioration affecting the structural soundness of the bridge or damage caused by impact or overloading

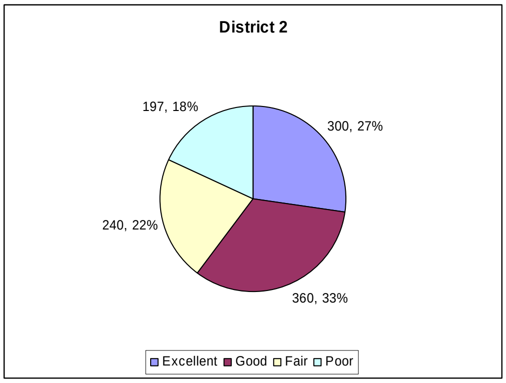
Total number of bridges/overpasses in district **623**

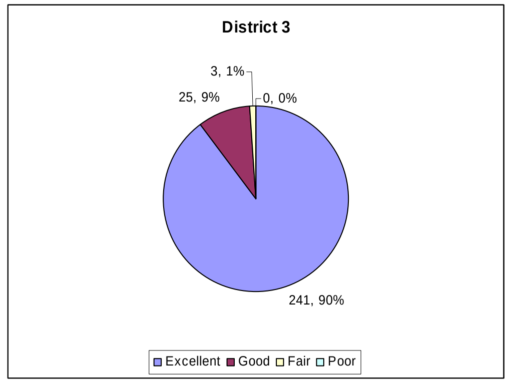
(continued on next page)

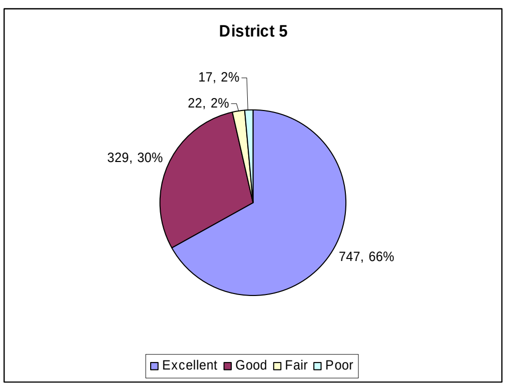
Please indicate the number of bridges in your district suffering each mechanism or manifestation of deterioration. A particular bridge may exhibit signs of more than one type of deterioration or none at all. Therefore, your total number in this section may not add to your total number of bridges.

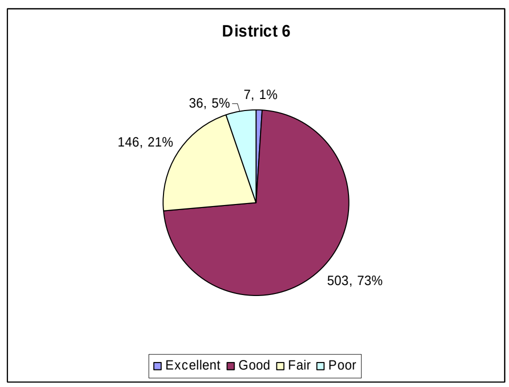
Corrosion (Spalling, Delamination, Cracking, etc.)	**80**
Cement Paste Hydrolysis (Efflorescence)	**225**
Light/Medium Scaling	**67**
Heavy/Severe Scaling	**0**
Pop-outs (Alkali-Silica Reaction)	**134**
Sulfate Attack	**0**
Construction Defects (Honeycombs, etc.)	**8**
Overload Damage	**14**
Collision Damage	**7**
Severe Wear	**6**

Bennie G. Holmes, Supervisor
Keith Alexander, Bridge Inspector — February 26, 2004
Signature — Date









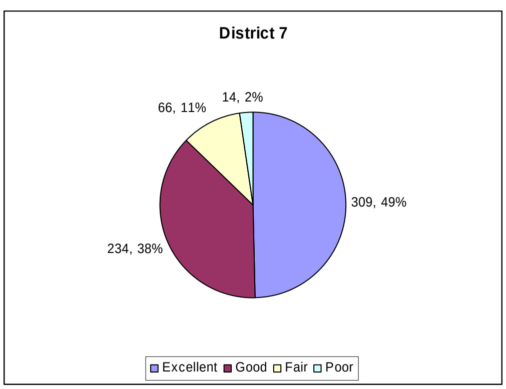


Figure 2.1 *Results of survey about service condition of bridges in five districts in Mississippi.*

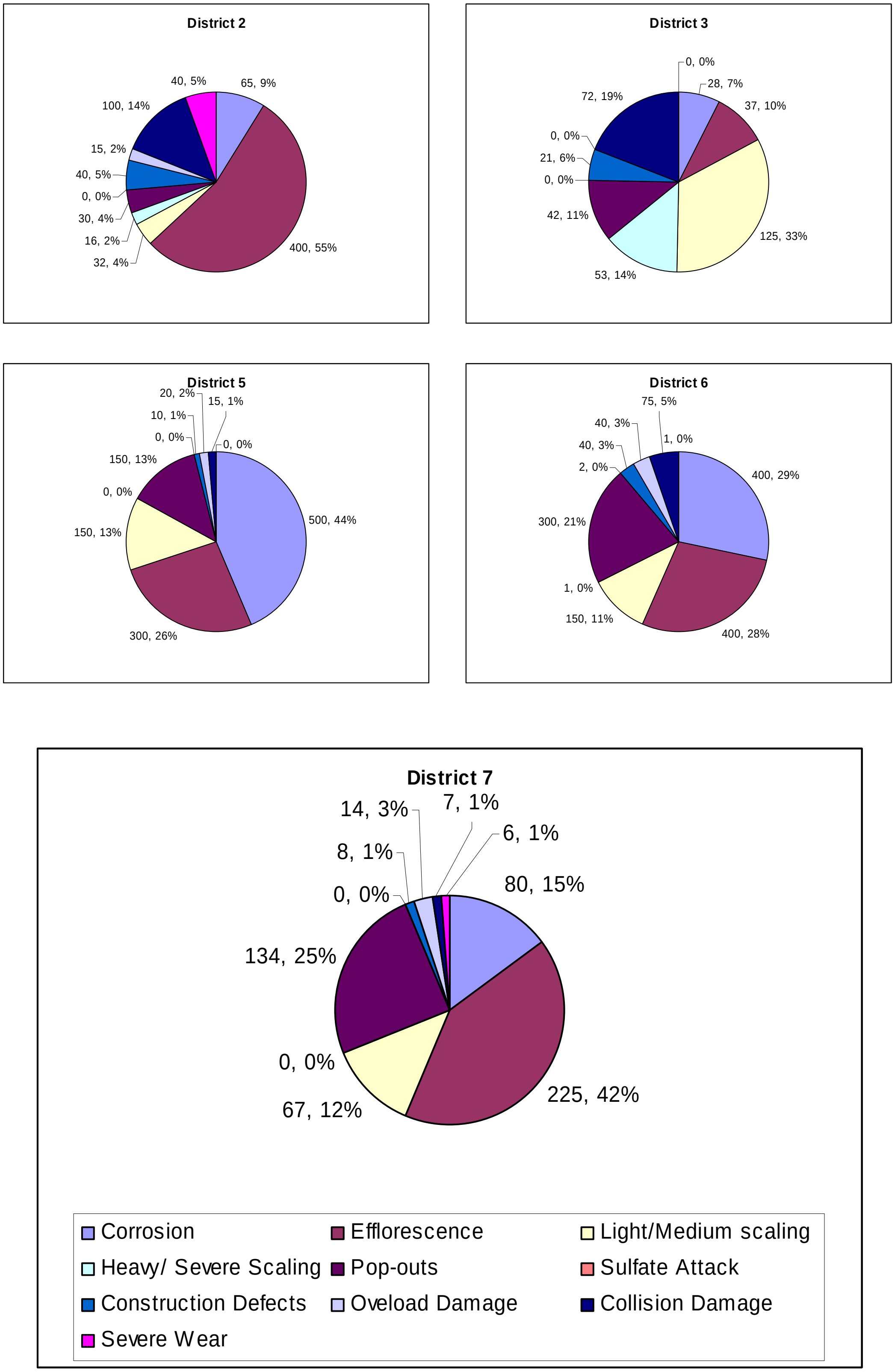


Figure 2.2 *Results of survey about main causes of deterioration of bridges in five districts in Mississippi*

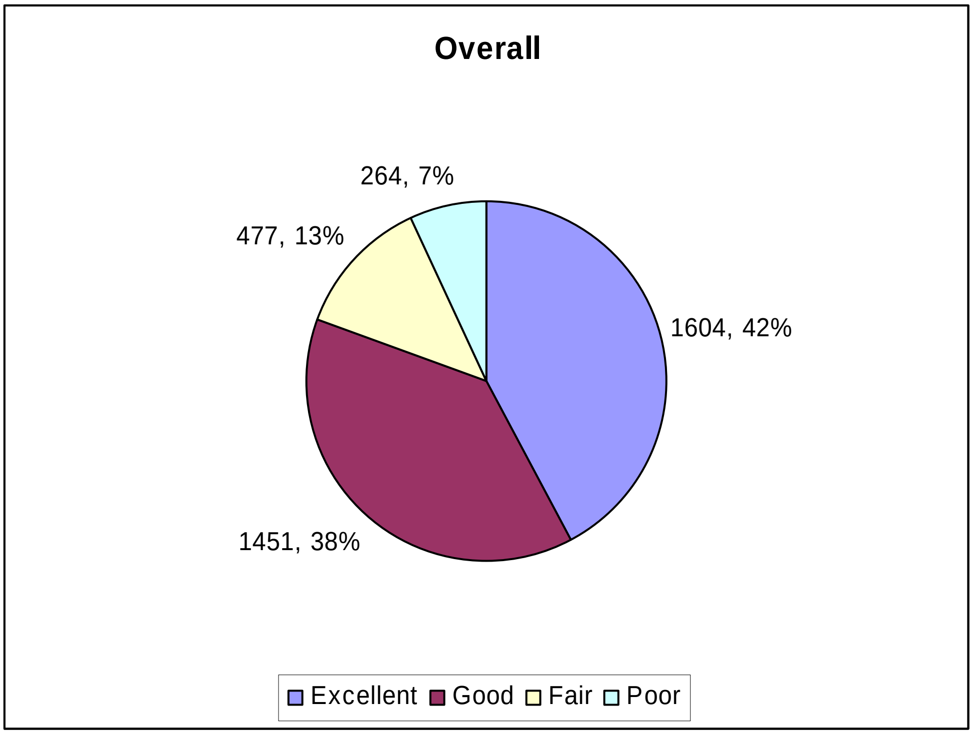


Figure 2.3 *Overall service condition of bridges in Mississippi*

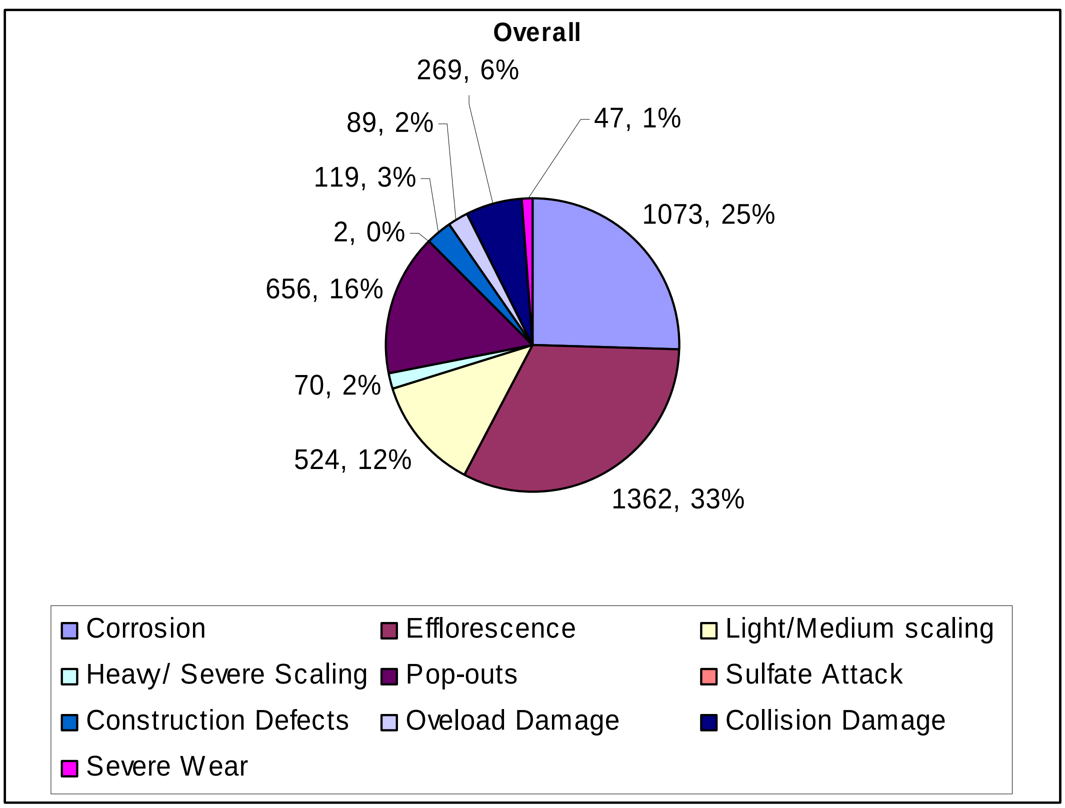


Figure 2.4 *Main causes of deterioration of bridges in Mississippi.*

CHAPTER 3

ATLAS ON CONCRETE DETERIORATION IN MISSISSIPPI

3.1 Manifestation of Concrete Deterioration

3.1.1 Cracking

Cracking is a common manifestation of concrete deterioration which can be caused by a variety of factors. Cracks which are found in bridges and overpasses are generally described as structural or nonstructural. Structural cracks are caused by both dead and live load stresses, which can lead to eventual failure of the structure. Flexure structural cracks are vertical and begin in areas of maximum tension or moment. Shear structural cracks are diagonal and are usually found in the web of a member. They may begin at the bottom and move diagonally toward the center of the member.

Nonstructural cracks can be caused by thermal expansion and contraction of concrete, contraction of the concrete during the curing process, or temperature gradients within massive sections of concrete. Also, the presence of rust stains around nonstructural cracks normally indicates corrosion of steel reinforcements in a concrete member. These cracks generally do not affect the load-carrying ability of a member, but may lead to higher susceptibility to other types of deterioration.

Figure 3.1.1 *Non-structural cracking due to thermal expansion and contraction observed in MDOT District I.*

Figure 3.1.2 Non-structural *cracking leading to efflorescence observed in MDOT District I.*

3.1.2 Scaling

Gradual loss of surface mortar and aggregate over an area is known as scaling. Scaling is classified as light, medium, heavy, and severe. Light scale is the loss of surface mortar up to ¼ inch deep exposing coarse aggregates. Medium scale is the loss of surface mortar from ¼ inch to ½ inch deep with mortar loss between the coarse aggregates. Heavy scale is the loss of surface mortar from ½ inch deep to 1 inch deep clearly exposing coarse aggregates. Severe scale is the loss of surface mortar greater than 1 inch deep where coarse aggregate particles are lost and reinforcing steel is exposed.

Figure 3.1.3 *Light scale observed in MDOT District V.*

3.1.3 Delamination

Delamination is the separating of concrete layers at or near the outermost layer of reinforcing steel. Delamination is caused by the expansion of corroding reinforcing steel and can lead to severe cracking. Rust can occupy up to ten times the volume of the corroded steel which it replaces.

Figure 3.1.4 *Delamination and spall (due to corrosion) observed in MDOT District I.*

Figure 3.1.5 *Non-structural cracking and loss of mass due to corrosion observed in MDOT District V.*

3.1.4 Spalling

Spalling occurs when a delaminated area completely separates from a member. The roughly circular or oval depression left is known as a spall. Friction from thermal movement can also cause spalling in addition to corrosion.

Figure 3.1.6. *Spall observed in MDOT District V.*

Figure 3.1.7. *Spall observed in MDOT District II.*

3.1.5 Efflorescence

Efflorescence is the result of hydrolysis of cement paste components in concrete. Efflorescence is indicated by the presence of white deposits on the concrete, usually on the underside of bridges and overpasses. Efflorescence indicates that the water used in the concrete mixing process was contaminated.

Figure 3.1.8 *Efflorescence observed in MDOT District I.*

Figure 3.1.9 *Efflorescence observed in MDOT District I.*

3.1.6 Construction Defects

This includes consolidation issues such as rock pockets, honeycomb voids, bug holes, and sand streaks which may result from improper vibration, dry mix without Super P's, over-watered mix, improper rebar spacing, or improper aggregate selection.

Hollow spaces or voids within concrete are known as honeycombs. They are caused during construction when improper vibration results in the separation of coarse aggregates from the fine aggregates and cement paste.

Figure 3.1.10 *Examples of construction defects.*

Another source of construction defects is related to insufficient concrete cover which may be caused by shift or cage shift, improper fabrication of steel, or improper placement of forms.

Figure 3.1.11 *Example of construction defects (insufficient concrete cover).*

3.1.7 Pop-Outs

Pop-outs are a result of alkali-silica reactions taking place in concrete. Conical fragments break out of the surface of the concrete leaving small holes. Shattered aggregate particles will usually be found at the bottom of the hole.

Figure 3.1.12 *Pop-outs observed in MDOT District II.*

3.1.8 Wear

Vehicular traffic causes wear on bridge decks throughout the life of the structure.

Figure 3.1.13. *Wear observed in MDOT District VI.*

3.1.9 Collision Damage

Vehicular collisions can cause severe damage to bridges and overpasses.

Figure 3.1.14. *Collision damage observed in MDOT District VI.*

3.1.10 Overload Damage

Overloading of bridges will result in structural cracking due to excessive vibration or deflection.

Figure 3.1.15 *Shear Crack observed in a bridges deck in MDOT District VI.*

3.2 Mechanism & Causal Factors of Concrete Deterioration

3.2.1 Corrosion

Steel reinforcement is added to concrete to increase its tensile strength. Steel is a product of naturally-occurring iron ore. A great amount of energy is required to convert iron ore to steel. Without proper protection, the process reverses, and oxidation occurs.

Corrosion is simply the process by which steel tends to return to its natural, oxidized state (Figure 3.2.1).

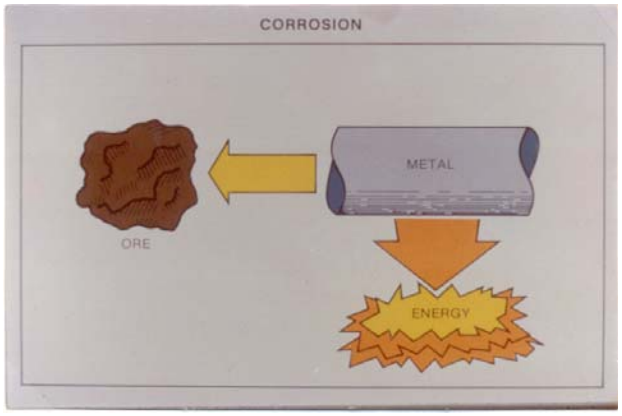


Figure 3.2.1 *Forms of iron in nature.*

Due to the chemistry of concrete mix, reinforcing steel embedded in the concrete is normally protected from corrosion. The high alkaline environment of concrete should cause a tightly adhering film to form on the steel (Figure 3.2.2).

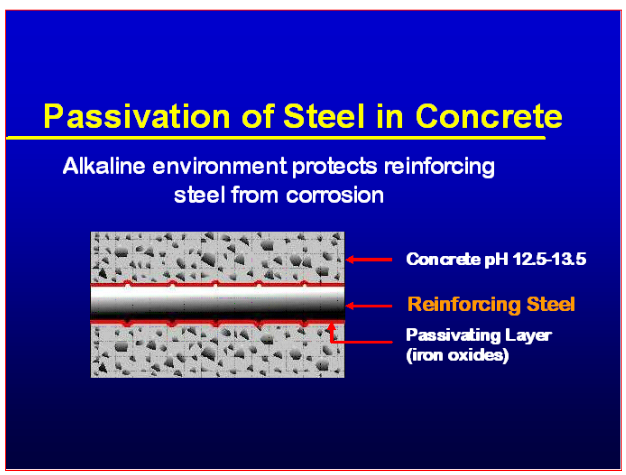


Figure 3.2.2 *Diagram of passivation of steel in concrete under normal conditions [91].*

However, the intrusion of chlorides, which enables water and oxygen to attack the reinforcing steel, eliminates the protective layer. Rust (iron oxide) is formed as a result (Figure 3.2.3).

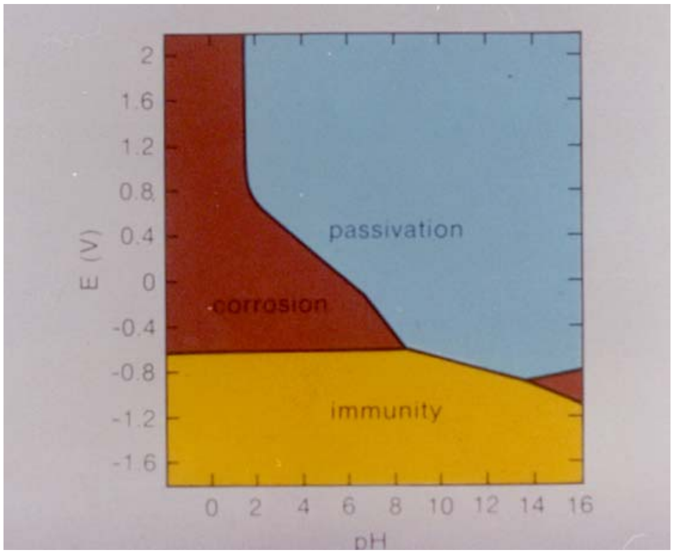


Figure 3.2.3 *Steel-Water Pourbaix Diagram.*

The passivating layer over the reinforcement is, however, broken when carbon dioxide enters the concrete and reaches the steel-concrete interface. This is called carbonation. Once this happens, steel starts rusting.

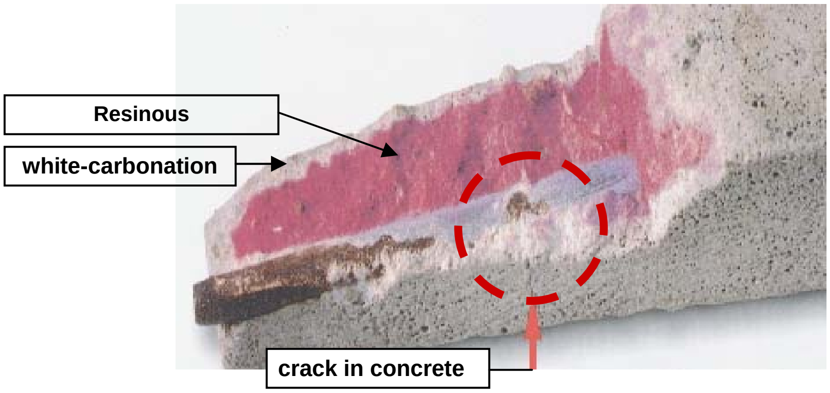


Figure 3.2.4 Carbonation of Concrete

Another powerful destroyer of the steel passivating layer is the chloride salt. With sufficient chloride present in concrete, the reinforcement is unprotected against corrosion. Moisture and oxygen, important elements in the corrosion process, usually penetrate through the concrete cover (Figure 3.2.5).

Figure 3.2.5 *Salt is the major cause of the corrosion process.*

Chloride ions are introduced into the concrete by marine spray, industrial brine, deicing agents, and chemical treatments. These chloride ions can reach the reinforcing steel by diffusing through the concrete or by penetrating cracks in the concrete (Figure 3.2.6).

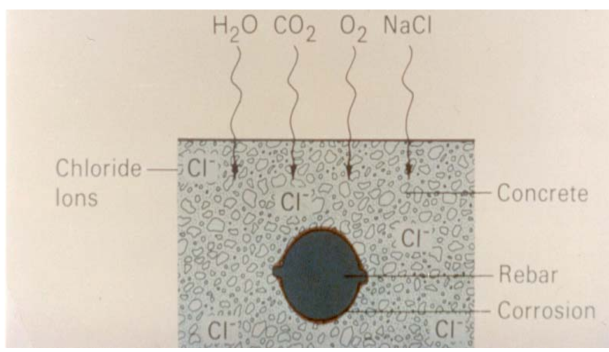


Figure 3.2.6 *Diagram of reinforced concrete saturated with chloride ions.*

Corrosion of steel in concrete is an electrochemical process. Therefore, electrochemical potential must be generated to form corrosion cells. This can occur when two dissimilar metals are embedded in concrete, such as steel rebars and aluminum conduit pipes, or significant variations exist in surface characteristics of steel (Figure 3.2.7).

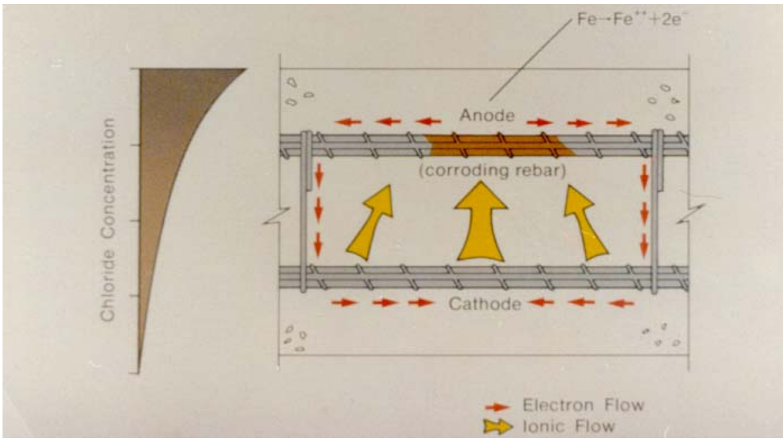


Figure 3.2.7 *Diagram of corrosion cell*.

Also, when differences in concentration of dissolved ions in the vicinity of steel exist, such as alkalis, chlorides, or oxygen, electrochemical potential may be generated. As a result of these instances, one of the two metals, or a part of the metal if only one is present, becomes anodic and the other cathodic Figures 3.2.8 and 3.2.9.

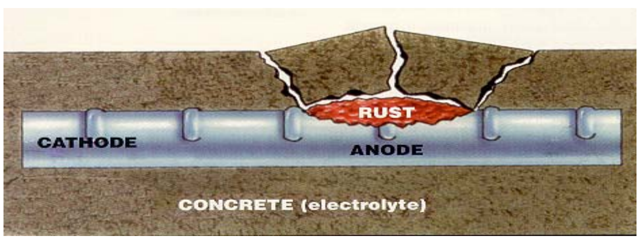


Figure 3.2.8 *Diagram of corrosion of steel reinforcement in concrete.*

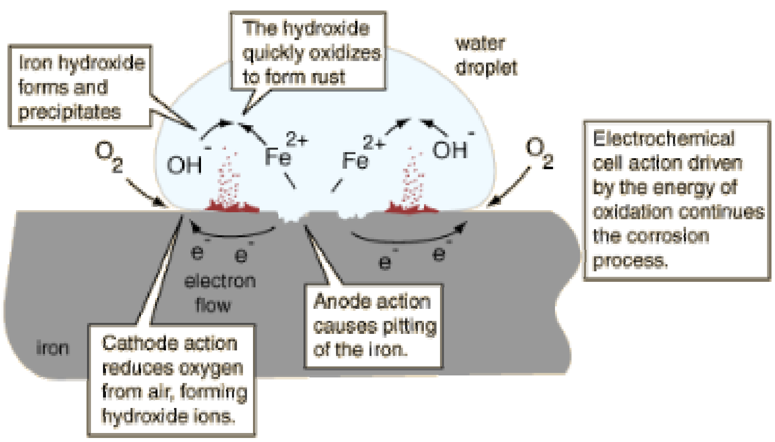


Figure 3.2.9 *Diagram of rust formation on steel reinforcement in concrete [92]*.

The steel is oxidized or ionized at anode and attracts the hydroxyl ions forming at the cathode. The oxidized steel combines with hydroxyl ions to form ferrous hydroxide or rust (Figure 3.2.10).

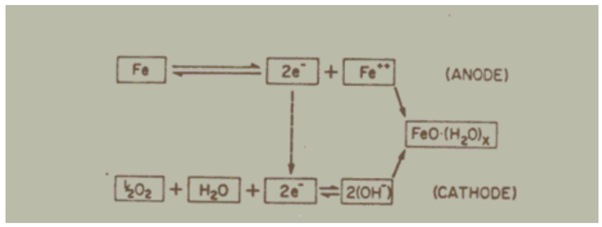


Figure 3.2.10 *Chemical equation for corrosion process.*

The rust occupies a much larger volume than the original steel and causes the build up of bursting forces at the surface of the reinforcement (Figure 3.2.11). Because concrete is weak in tension these bursting forces quickly cause the concrete to crack parallel to the reinforcement direction and eventually, to spall away from rebars (Figure 3.2.12).

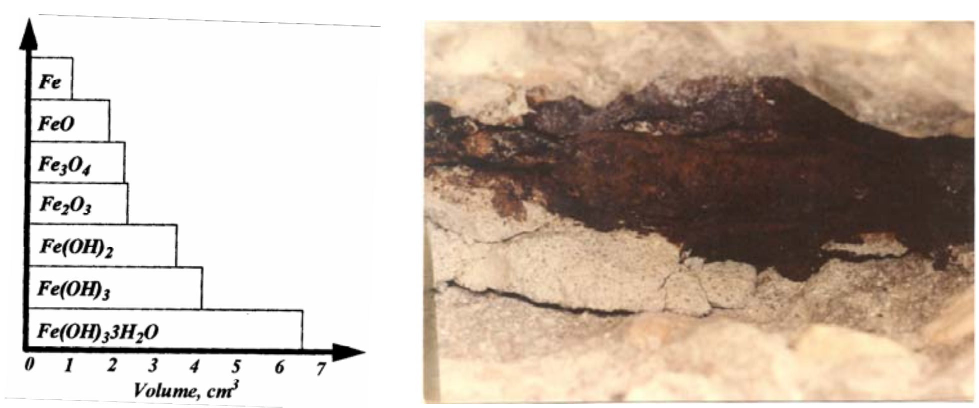


Figure 3.2.11 *Diagram of the progression of rust over time during corrosion.*

Figure 3.2.12 *Example of corrosion of reinforcement in a bridge deck in District 1.*

3.2.2 Hydrolysis of Cement Paste Components (Efflorescence)

Normally, water from lakes, rivers, or ground water is used when mixing concrete. Hard water contains chlorides, sulfates, and bicarbonates of magnesium and calcium. These components do not attack the constituents of Portland cement paste.

When water from rain, condensation, or melting snow and ice is introduced into the concrete mix, contamination can occur. Pure or soft water from these sources contains little or no calcium ions. Therefore, they tend to hydrolyze or dissolve the calcium containing products in the concrete mix. Eventually, the process leaves behind silica and alumina gels with little or no strength. Efflorescence occurs as white calcium deposits leach from the concrete (Figure 3.2.13). This is the most common sign of deterioration in Mississippi.

Figure 3.2.13. *Example of efflorescence in District 1.*

3.2.3 Cation-Exchange Reactions

The formation of soluble calcium salts in concrete can cause serious weakening of concrete. Ammonium chlorides and ammonium sulfates found in fertilizer and agricultural products are one cause of the formation of calcium salts. For example, the following reaction can occur:

$$2NH_4Cl + Ca(OH)_2 \Rightarrow CaCl_2 + 2NH_4OH.$$

Carbonic acid attack is one type of cation-exchange process that occurs due to the acidity of naturally occurring water. Carbon dioxide is the cause of the acidity and is found in significant concentration in mineral waters and seawater. The following reactions occur:

$$Ca(OH)_2 + H_2CO_3 \Rightarrow CaCO_3 + 2H_2O$$

$$CaCO_3 + CO_2 + H_2O \Leftrightarrow Ca(HCO)_3$$

Magnesium ion attack is the most aggressive form of deterioration caused by cation-exchange reaction. On prolonged contact with magnesium solutions, calcium silicate hydrate, a principal component of Portland cement paste, loses calcium ions, which are thereby replaced with magnesium ions. The end result is the leaching of calcium products from the concrete, indicating a loss of cementitious characteristics.

3.2.4 Sulfate Attack

Agricultural soils and waters and the decay of organic matter can result in sulfate attack. Due to the presence of magnesium and alkali sulfates in these materials, SO_4 concentrations are higher than normal.

The form of the deterioration process depends on the concentration and source of sulfate ions in water, and the composition of the cement paste in concrete. Either expansion of concrete or a progressive loss of strength and mass will occur. Sulfate attack can be classified by four degrees of severity: negligible attack, moderate attack, severe attack, very severe attack. Very little sign of sulfate attack was observed in Mississippi. Thus, I will not discuss this mechanism in more detail.

3.2.5 Alkali-Silica Reaction

Expansion and cracking around each piece of aggregate can result from chemical reactions involving alkali ions from Portland cement, hydroxyl ions, and silica that may be present in the aggregate. Raw materials used in Portland cement manufacture account for the presence of alkalines in cement (0.2 to 1.5% Na_2O).

Depending on the alkali content of the cement, the pH of the pore fluid is generally 12.5 to 13.5. This pH means the liquid is strongly alkaline and some acidic rocks do not remain stable on long exposure. Aggregates composed of silica, siliceous minerals, opal, obsidian, cristobalite, tridymite, chalcedony, and cherts are particularly vulnerable. This process of deterioration by alkali-silica reaction is also known as the *cancer of concrete*.

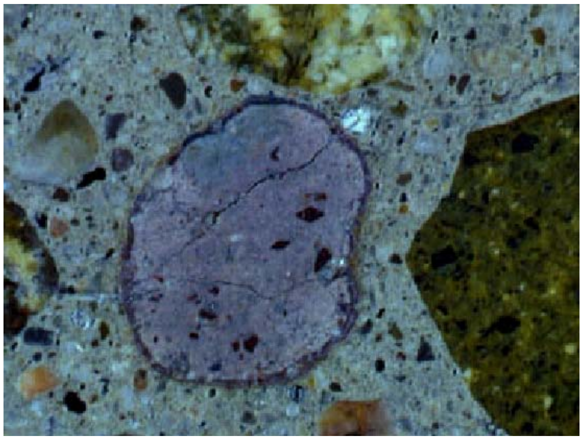


Figure 3.2.14. *Gel formation around aggregate due to alkali-silica reaction [93].*

Figure 3.2.15. *Aggregate expansion due to alkali-silica reaction [93].*

Figure 3.2.16. *Result of aggregate expansion on bridge supports.*

3.3 Examples of cases of concrete bridge deteriorations in Mississippi

3.3.1 Examples of cases of concrete bridge deteriorations in District 1

(a)

(b)

(c)

(d)

(e)

(f)

Figure 3.3.1. *Efflorescence.*

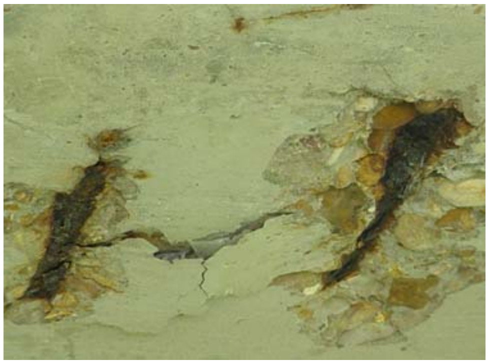


Figure 3.3.2. *Loss of concrete mass due to corrosion.*

Figure 3.3.3. *Loss of concrete mass due to efflorescence.*

Figure 3.3.4. *Corrosion of reinforcement.*

Figure 3.3.5. *Corrosion of reinforcement*

Figure 3.3.6. *Cracking due to thermal expansion and contraction.*

Figure 3.3.7. Cracking due to corrosion.

Figure 3.3.8. *Crack repair*.

Figure 3.3.9. *Loss of concrete mass.*


(a)


(b)

Figure 3.3.10. *Delamination and spalling of concrete due to corrosion resulting in exposure of steel reinforcement.*

3.3.2 Examples of cases of concrete bridge deteriorations in District 2

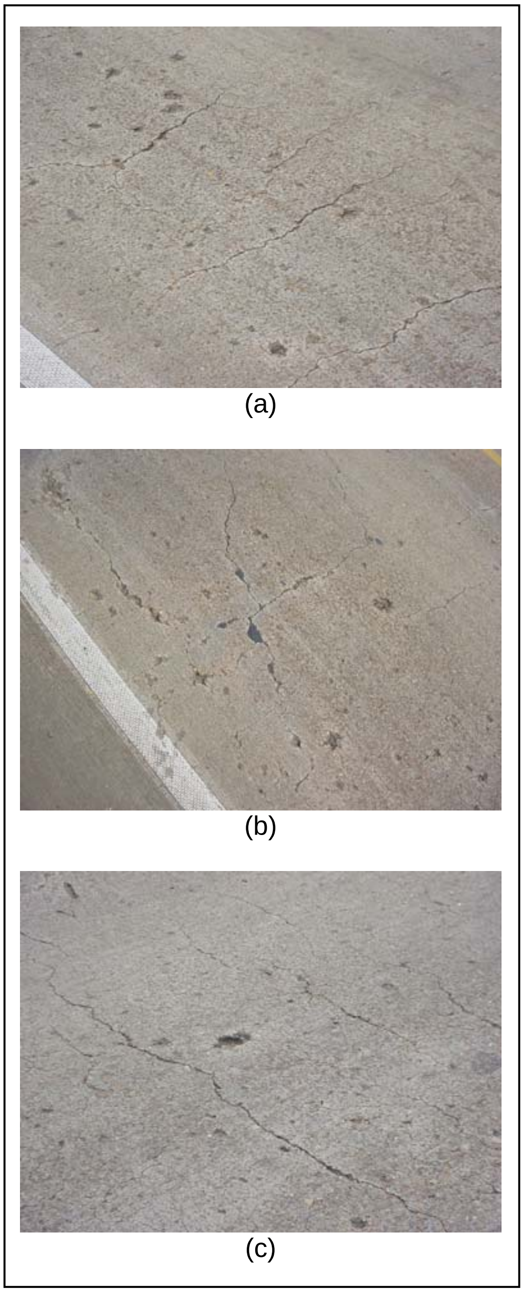


Figure 3.3.11. *Cracking on road deck due to thermal expansion and contraction*

Figure 3.3.12. *Underside of bridge deck showing initial signs of delamination.*

Figure 3.3.14 *Joint failure due to traffic impact.*

Figure 3.3.15. *Pop-outs on bridge rail.*

Figure 3.3.16. *Severe spall.*

Figure 3.3.18. *Sever scale.*

Figure 3.3.17. *Corrosion*

3.3.3 Examples of cases of concrete bridge deteriorations in District 3

(a)

(c)

(b)

(d)

3.3.19. *Severe scale.*

(a)

(b)

3.3.20. *Medium scale.*


(a)


(b)


(c)


(d)


(e)


(f)


(g)


(h)

Figure 3.3.21. *Severe scaling and spalling.*


Figure 3.3.22. *Light Scale*


Figure 3.3.23. *Medium Scale*


Figure 3.3.24. *Wear due to traffic.*


Figure 3.3.25. *Bridge deck over the Quiver River*.


Figure 3.3.26. *Medium scale.*


Figure 3.3.27. *Scale and wear.*


Figure 3.3.28. *Discoloration of bridge deck due to underlying corrosion.*


Figure 3.3.29. *Spalling and discoloration of bridge deck due to underlying corrosion.*

Figure 3.3.30. *Scaling and discoloration of bridge deck due to underyling corrosion.*

Figure 3.3.32. *Discoloration of bridge deck indicating corrosion of steel reinforcement.*

3.3.5 Examples of cases of concrete bridge deteriorations in District 5

Figure 3.3.33. *Efflorescence.*

Figure 3.3.34. *Spalling.*

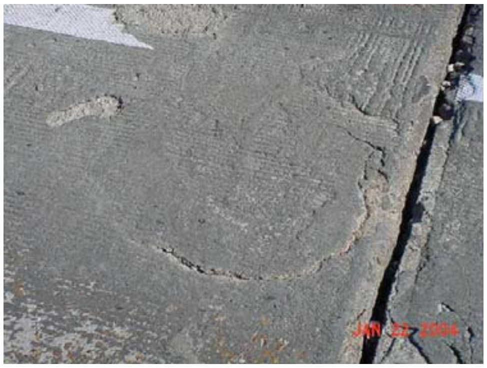


Figure 3.3.35. *Scale and joint failure*.

Figure 3.3.36. *Spalling.*

Figure 3.3.37. *Loss of mass due to Corrosion of reinforcement.*

Figure 3.3.38. *Severe loss of mass due to corrosion of reinforcements.*

Figure 3.3.39. *Delamination*.

Figure 3.3.40. *Cracking due to Underlying corrosion.*

Figure 3.3.41. *Efflorescence.*

Figure 3.3.42. *Exposure of steel reinforcement.*

Figure 3.3.43. *Severe exposure of steel reinforcement.*

Figure 3.3.44. *Delamination.*

3.3.6 Examples of cases of concrete bridge deteriorations in District 6

Figure 3.3.45. *Cracking due to thermal expansion and contraction.*

Figure 3.3.48. *Structural shear crack due to overloading (under-designed case)*

Figure 3.3.46. *Severe cracking and separation of bridge deck from roadway.*

Figure 3.3.49. *Leaching of Calcium Hydroxide.*

Figure 3.3.47. *Discoloration and staining indicating corrosion.*

Figure 3.3.50. *Severe spalling.*

Figure 3.3.51. *Collision damage.*

Figure 3.3.53. *Cracking and separation of bridge joint repair from roadway.*

Figure 3.3.52. *Light scale and spalling.*

Figure 3.3.54 *Light scale and wear.*

CHAPTER 4

GUIDELINE FOR THE SELECTION OF SUITABLE REPAIR MATERIALS AND METHODS

Concrete structures are inherently durable and usually, if properly designed and constructed, require minimum repair and maintenance. However, if concrete is placed in a harsh environment then it will deteriorate with time which will require remedial treatment and repair.

The excessive deterioration of concrete structures in aggressive environments opens a potential field for the usage of repair materials. A whole range of resinous and cementitious repair materials and several relatively new repair techniques have been proposed by materials manufacturers and construction contractors and are being aggressively promoted for usage in the market. However, neither the repair materials nor the application of repair techniques to concrete structures has been adequately investigated, especially in aggressive climatic conditions.

The goal of a concrete repair is to make a successful repair, which means restoring the deteriorated area to near original condition and all of this must be done with as little delay to traffic as possible. It is preferable to use a repair material that is identical to that of the original concrete, however this is not possible and the repair material will differ from concrete. This being said, the concrete repair area will act as a composite system. This composite system consists of three components:

1. The original concrete substrate.
2. An interface between the original substrate and the new repair material.
3. The new repair material.

4.1 Preparation for Concrete Repairs

Preparation for repairs is unanimously regarded as the most important step [16,94,95] and several repair failures have been traced to inadequate preparation [94]. This phase involves:

(a) removal of all deteriorated concrete,

(b) cleaning of steel reinforcement of all corrosion products, and

(c) preparing the concrete surface for priming treatment.

(a) Removal of Deteriorated Concrete

Removal of spalled and loose concrete is carried out using scrabbles, chisels and hammers or any other suitable mechanical means, manual or pneumatic. Use of high frequency chipping pneumatic or electric hammers of less than 10-pound capacity may be used without shattering sound concrete [95]. Heavy duty pneumatic hammers are not favored. Defective and contaminated concrete is also removed by the use of percussion tools, grit blasting (wet or dry) or high velocity water jetting [96,97]. Perkins [98] considers that high velocity water jetting offers the best chance for a quick and clean job in terms of concrete removal provided the presence of water is accepted. Thermic lance [99] can also be used with considerable efficiency and success if cutting through reinforcement is acceptable. For repair of small concrete spalls, it is recommended [94] that concrete be cut to a depth consistent with the type of repair. In a review article, “Taking the tedium out of cutting” [100] Sclippa describes a whole range of diamond tipped tools and devices for grinding, safety grooving, large-radius cuts and flame cutting of concrete.

(b) Cleaning of Steel Reinforcement

Cleaning of reinforcement follows the removal of all loose and defective concrete. The exposed bars should be cleaned of rust and corrosion products. This operation is particularly important in the case of chloride-afflicted concrete, where chlorides will be present in the rust. The required degree of cleaning has not been completely defined. Throughout the literature, grit blasting is considered one of the most effective methods of cleaning the steel [100-102] and has the advantage of being able to reach the back of the bars. Wire brushing is not a very effective method and tends to polish rather than remove scale [103].

(c) Preparing Concrete Surfaces for Priming and Bonding

Concrete surfaces to be repaired or on which primer coating is to be applied should be newly exposed parent concrete free of loose unsound materials.

Grinding is a useful way to clean small areas, particularly if the cleaned surface must be smooth [94, 102]. Scarifies will remove thick overlays of dirt or weakened concrete but they usually leave the surface somewhat rough. They may also weaken some of the aggregate by impact and consequently have to be followed by water blasting or vacuum cleaning [102] to remove the loosened particles.

Probably the best way to remove laitance, dirt, efflorescence and weak surface material is to clean by wet or dry sandblasting or high pressure water jet followed by vacuum cleaning. The air compressor for sandblasting is usually equipped with efficient oil and water traps so that the sand is free of oil particles. Only clean water should be used for wet sandblasting. Blast cleaning should continue until all weak surface particles are removed, leaving small aggregate particles exposed. When mechanical abrasion cannot be used due to dust hazards or other environmental limitations, the surface may be etched with acid.

4.2 Concrete Repair Techniques

The present state of the art on repairing and strengthening existing structures employs systems which have largely been developed through experience and these are empirical in nature.

The types of deterioration and repair work relevant to Mississippi have been identified and a review of the possible repair techniques and systems related to the identified problems has been carried out.

4.2.1 Replacing or Adding Reinforcing Steel

Rebar that has been excessively corroded has to be replaced. This is generally accomplished by removal of the damaged portions and replacing with new steel welded in place. Generally full penetration butt welding is preferred though lap welding may be used in some cases. In some cases, conventional lap joints are made and in those cases where the reinforcing is in tension only, standard mechanical splices need to be used. In cases where sections to be strengthened are interrupted by existing columns or beams, continuing the new reinforcing in holes drilled through the existing element is desirable. If the reinforcement is depleted only partially, new bars are sometimes added along the length of the old cleaned bars.

Where it is not possible to penetrate the element such as in corners or at termini, or where additional shear resistance is required, reinforcing steel dowels are secured in drilled holes. Dry pack, non-shrink cementitious grout and epoxy resin materials need to be used for this purpose. The epoxy resin materials have been proven most suitable [104-

106] as they require a smaller hole, minimizing possible interference with existing reinforcing, as well as being more economical. Tests have shown that epoxy set dowels properly installed will retain their full yield capacity when embedded approximately ten times their diameter.

4.2.2 Small Spall Repair

Relatively minor spalls are routinely repaired by shotcrete, epoxy-sand mortar, non-shrink cementitious grouts, or standard cement-sand mortar or dry-pack [32,107,108]. Where non-shrink grout or cement-sand mortars are used, bonding agents of moisture compatible epoxy, polymer emulsion, or neat cement-water paste are sometimes used. It is important that all loose material is removed from such areas and the surface properly roughened and free of contaminants prior to patching.

Several repair and material systems may be used for the reinstatement of the concrete section after the removal of unsound spalled concrete. The replacement is either made with sprayed concrete, resin or polymer modified concrete, conventionally poured concrete, or pre-placed aggregate concrete.

(i) Sprayed concrete (Shotcrete): Sprayed concrete [109,110] has been used in a number of repair situations; however, the nature of the method is such that the original surface finish and profile will not usually be produced. Basically, two systems exist; the wet method where all mix constituents including the water are premixed and transported to the gun via compressed air, and the dry method where the dry constituents are transported to the ejection nozzle at which point water is injected. The latter method is the more widely used.

Normally low water contents are used and compaction is achieved by the velocity of the particles, control of the water content rests entirely with the operator and hence the method is particularly sensitive to the skill, or otherwise, of this person.

Where heavily reinforced sections are to be repaired, care needs to be taken to avoid shadowing in the lee of the spray gun.

The advantages of this system are that high strength low permeability concrete can be produced without formwork, which is particularly important for repair of the underside of beams and slabs. The original profile of the beam or slab can be restored by passing a screed wire over the applied concrete.

(ii) Conventional Concrete: Conventional concrete may be successfully used to replace defective concrete especially where the areas of defective concrete are significant [105-112]. The composition of cast-in-place –concrete is more uniform than that of gun-applied mixes. With the use of super-plasticizers, the w/c can be reduced to 0.38 with excellent workability.

The greatest drawback of conventional pours is that the uppermost space, for example, a beam located under a slab, cannot be filled by gravity. These spaces must later be injected with epoxy compounds.

(iii) Polymer Modified Concrete/Mortar: Replacing up to 33 percent of mixing water of a conventional concrete mix by a latex emulsion imparts many improved properties to the modified concrete [113-114]. These include improved strength, excellent bond with old concrete, reduced shrinkage and reduced permeability. The modifying admixture can be added to a low slump mix to give very high workability without the normally associated problems of low strength and high permeability [113]. Styrene butadiene rubber(SBR), acrylic and modified acrylic latexes are all widely used as admixtures in repair concrete/mortars.

(iv) Epoxy Modified Concrete/Mortar: Unlike cement based repair systems, whose high alkalinity helps prevent steel reinforcement corrosion by passivation, the protection afforded by resin concretes/mortars is achieved by encapsulating the steel reinforcement with an impermeable 'macro' coating which exhibits excellent adhesion to both the steel and concrete substrate [115]. This protective mortar/coating will give good long-term protection of steel reinforcement at thicknesses far less than is possible with cementitious repair materials. Epoxy resin concretes/mortars are most widely used in concrete repairs. Polyester resin and acrylic resin based mortars are also used, generally for small area repairs where their very rapid development of strength is required.

Comments have been made that epoxy resin mortar repairs have not always proved durable even in the short term. It is, therefore, most important to understand that the generic name epoxy resin covers a very diverse range of chemically and physically different polymers. To achieve good durable repairs, careful selection of the resin composition, and grading of the fillers appropriate to the application and service conditions is essential.

(v) Pre-placed Aggregate in Concrete: Pre-placed aggregate in concrete [32, 107, 108] is a variation of the concrete repair system which has the advantage that the coarse aggregate is in inter-particle contact, thereby eliminating segregation and settlement and minimizing drying shrinkage of the concrete. This method also requires no vibration during placement.

It possesses the added advantage of being suitable for use under water where the injected grout displaces the water, saving dewatering costs or the need for watertight formwork.

The principal disadvantage of the method is that the injected cement paste itself can be prone to bleeding, which will manifest itself as water lenses beneath the aggregate particles and at the concrete interface above, destroying bond at that point and providing a route for attack by the environment. This is usually overcome by using non-shrink grout mixes which are formulated to be workable but not to bleed.

4.2.3 Bonding Coats

The effectiveness of any repair method will largely depend upon its ability to achieve an effective bond with existing concrete. When applying conventional concrete, sprayed concrete or sand/cement repair mortars, bond is often a problem. In particular, where the repairs are to be carried out at high ambient temperatures, water loss at the interface between the repair material and the prepared concrete may prevent proper hydration of the cement matrix at this interface. The use of an epoxy resin or polymer latex bonding aid can assist in achieving a reliable bond. With an epoxy bonding system, specifically formulated for bonding green uncured concrete to cured concrete, a bond is achieved which is significantly greater than the shear strength of good quality concrete or mortar. In Europe, polymer latex bonding aids which are applied to the prepared concrete either as neat coats of latex or as slurries with cement are widely used since they are simpler to use than epoxy resin bonding aids and give a good tough bond. Cement/SBR mixes are traditionally and most widely used to bond fresh concrete to mature old concrete and this particular repair usage is well documented. To-my-knowledge, this is the main reason of failures of repairs in Mississippi.

4.2.4 Crack Repair

Perhaps the number one consideration in any remedial treatment is the repair of existing cracks. This is a technique which is rather well documented including the most comprehensive report entitled “Causes, evaluation and repair of cracks in concrete structures” by ACI Committee 244 [52]. This document also contains an extensive bibliography.

Cracks in concrete represent one of the most difficult problems in the repair and maintenance of concrete. Cracks often form as unintended movement joints, and designers frequently want to seal them and make them invisible.

It is important to be quite clear why it is required that a particular crack should be treated:

(a) to prevent water penetration,

(b) to protect the reinforcement,

(c) to prevent staining from material leached out, and

(d) to conceal the crack?

In many cases, more than one reason will apply. However, the importance of achieving the desired result must be considered against the difficulties. Cracks tend to vary in width with thermal and moisture changes, even when variations in applied loadings are not involved. Weathering tends to accentuate differences in surface absorption, which are inevitable at cracks.

Cosmetic disguising of cracks has been attempted with varying degrees of success by rubbing or brushing into the surface a mixture of cement and fine sand gauged with a clear polymer emulsion, for example an acrylic emulsion. Provided that subsequent movements are very small, an acceptable result may be obtained, although there is always the danger that careless workmanship will actually make matters worse, rather than better.

If prevention of water penetration at cracks is the requirement, the following methods are available: injecting chemically curing resins (e.g., epoxy resins); pouring in latex emulsions; cutting a surface chase and sealing with a mastic or sealant; and sticking a ‘bandage’ over the crack and painting the surface to try to conceal it. Each of these methods has its drawbacks.

Resins can be injected into relatively fine cracks, using techniques now available. Cracks down to 0.1 mm can be filled. Restoration of full structural properties is possible, provided that the causes of cracking have been removed. If they have not, fresh cracking is likely adjacent to the old.

Latex emulsions build up a latex deposit in the crack. If the crack does not subsequently move by more than about + 10% of the width when filled, they can provide a measure of protection against water penetration. However, cracks frequently move by

+50% or more and most material will have great difficulty in accommodating such movement.

Chasing and sealing cracks is usually technically satisfactory, but it clearly has acute aesthetic drawbacks. Cracks often do not form neat straight lines, which can make the chasing process very difficult.

'Bandaging' is usually only used when major concrete maintenance is required. It too has visual disadvantages.

It should be clear from this brief review of the difficulties of repairing cracks that there is considerable scope for research in this field. Before new techniques are devised work should only be undertaken when it is absolutely essential. The recent investigations of the influence of crack widths on reinforcement corrosion should lead to a reduction in the extent to which crack repairs are undertaken solely to protect reinforcement.

4.2.5 Honeycombs and Voids

Poorly consolidated concrete, resulting in a patch of 'honeycomb' is usually cut out and replaced, but sometimes a more immediate restoration may be required to avoid delaying subsequent operations. Patching with cementitious or resin mortars is often advocated, but is only likely to be satisfactory if the fault is confined to the surface. If there are deeper voids around the reinforcement, this surface patching will afford little protection and the long term result is likely to be rusting of the steel with resultant staining and possibly spalling of the concrete. Resin injection can provide an effective repair if there is good connection between the voided areas. It will not only protect the steel but can upgrade the weak 'honeycomb' material to the strength of dense concrete.

4.2.6 Strength With Bonded Reinforcement

If a structure requires strengthening, a technique which will cause minimum disruption and minimum change of profile is the external bonding of steel plate reinforcement [116-119]. This technique has been in use for nearly four decades but has not been very widely accepted. Earlier work in the U.K. [120] has quantified the increase in strength attainable as 100 percent the load required to produce the first visible crack, 40 percent in maximum load and an increase in stiffness of 190 percent, all related to the original performance of the beams as cast. Benefits of a similar order were obtained by plate-reinforcing beams which had already been cracked by overloading. Coupled with crack injection this offers a formidable and elegant means of restoring and upgrading a damaged structure.

4.2.7 Concrete Replacement with Fiber Reinforced Concrete/Mortar

The important shortcomings of cementitious materials are brittleness and low tensile strength. These inherent disadvantages can be considerably improved by incorporation of fibers [121,122].

The introduction of fibers into a cementitious matrix forms a material which exhibits higher tensile strength and toughness than those of the matrix alone. The fibers used in concrete and mortars include steel, glass and more recently, polymeric fibers.

Steel fiber reinforced concrete exhibits a higher flexural strength and fracture toughness than conventional concrete. Use is limited by the minimum thickness of application, 1.1/2-2 in., (about 45 mm) and difficult surface finishing. Fibers are introduced into the concrete by mixing either on site or in a plant and the concrete is placed by using standard placement techniques.

Polymeric fibers added to concrete can replace the secondary steel reinforcement in concrete slabs and provide some degree of fracture toughness. They may also control plastic and drying shrinkage-induced cracking. The low modulus of elasticity of polymeric fibers cannot provide any significant improvement of tensile or flexural strengths. Standard concrete placement techniques are employed with polymer fiber reinforced concrete.

The premix process introduces the fiber by mixing it with a cement mortar, which is then applied by trowelling, screening, spraying, or casting. The small quantities of fibers (typically ½ to 2 percent of the fiber by volume) control drying shrinkage-induced cracking by forming microcracks within the structure of the material.

The small amounts of glass fiber also improve the fracture toughness and tensile strength of the cementitious mortar, thus allowing thin layer applications (typically 1/8 to ½ in.). Water tightness, resistance to chloride penetration, hardness, and abrasion resistance are properties primarily controlled by the composition of the cementitious matrix and, therefore not fully dependent on the presence of the fibers.

A thin layer, waterproofing characteristics, breathability, compatibility with concrete, high abrasion resistance and long-term durability of suitably formulated glass fiber reinforced mortars make this group of materials particularly suitable for thin toppings on concrete slabs, bridge and parking decks, industrial concrete floors, and in waterproofing of concrete structures and other concrete repair. The use of glass fiber reinforced cement has been reported as a repair material both by patching [121] and spraying [122] techniques.

4.2.8 Rehabilitation and strengthening of deteriorated concrete with Fiber Reinforce Polymer (FRP).

Fiber reinforced polymer (FRP) materials are being used increasingly to retrofit concrete bridges and many DOT's have implemented or will be implementing several field projects. Retrofits on beams, pier caps, and decks typically involve bonding of FRP to concrete, and integrity of the bond is crucial for success. However, moisture and salt ingress into rehabilitated components has the potential to degrade the bond between FRP and concrete due to continued corrosion within the concrete or delamination of the FRP during freeze/thaw cycles.

4.3 Surface Treatment with Protective Coatings and Penetrating Sealers

At the end of a repair, it is often desirable to apply a sealing coat to seal both the repaired areas and the remainder of the structure for aesthetic reasons and for reducing the diffusion of oxygen and carbon dioxide which accelerate the corrosion of reinforcement. In reinforced concrete structures, where chlorides are already present throughout the concrete (above 0.04% chloride on cement content), there are no practical methods of totally arresting reinforcement corrosion. However, the use of coatings which reduce the ingress of oxygen and moisture have been found in many instances to reduce the rate of deterioration to an extent that further corrosion/spalling could be dealt with on a regular maintenance basis [123-124]. When applying protective coatings, it is essential that concrete surfaces are thoroughly clean and sound.

The choice of the protective coating system is quite wide and various compositions have been used to coat concrete including bituminous coatings, chlorinated rubber, polyvinyl copolymers and terpolymers, acrylics (reactive, solvent based and water based), polyurethanes and epoxy resins. Such coatings, if free from defects (crack, pinholes, etc.) prevent the passage of water or aqueous salts in liquid or mist form and have low permeability to water vapor, carbon dioxide and oxygen. Long term durability depends upon a number of factors including chemical composition of the binder, precise formulation of the coating, total thickness and application techniques.

Penetration sealers which reduce chloride ingress include acrylic resin solutions, water repellent silicone resins and certain types of silane resins, epoxies and polyurethanes. Providing the materials have filled the pores within the surface of all of the concrete as intended, they should give good long term durability. However, conventional silicone resin types which function purely by making the pores water repellent seldom last more than a few years. The alkyl silanes function in the same manner and they are more durable than silicone resins. The molecular size or silane penetrants are important as it significantly influences the depth of penetration into the surface of the concrete.

4.4 Repair Material

There are many concrete repair materials in today's market that are very capable of producing successful concrete repairs. However, before a successful repair can be made a general material selection process is needed to insure that the best repair material is selected. This selection process first involves determining the project objectives. These objectives are:

- **<u>Causes of deterioration:</u>**

  Determining the causes of deterioration is the first step in selecting the proper material for the repair. The information for this section will come from the other paper on concrete deterioration.

- **<u>Owner (MDOT) requirements:</u>**

  This step of the selection process is simply to make sure that the scope of the project is properly understood. Some of the items to consider are project budget, appearance, expected service life, and any structure utilization needs during rehabilitation. These are some of the basic considerations, which must be taken into account before any other decisions are made about repair materials.

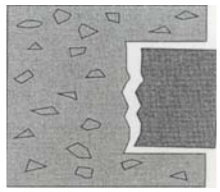
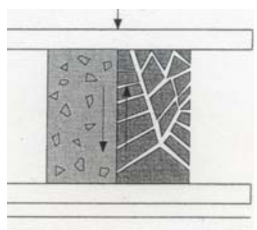
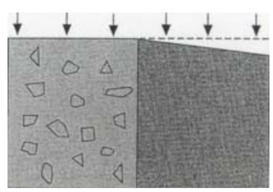
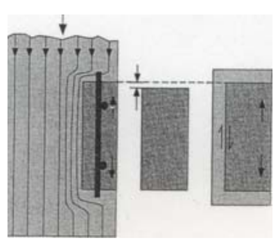
- **<u>Service conditions:</u>**

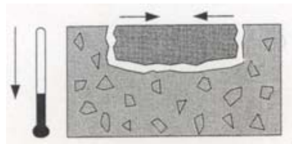
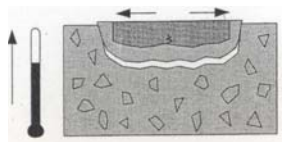
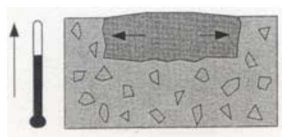
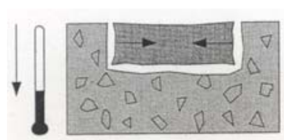
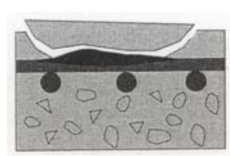
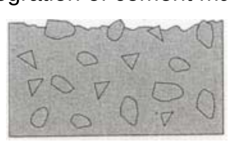
  Determining service conditions is important because it allows you to determine the physical and chemical properties needed in the repair material depending on the different load factors that the bridge will see. These load factors include weather, chemical environment, and live loads.

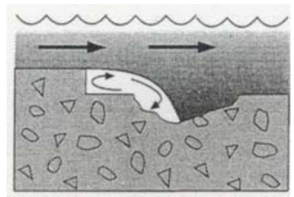
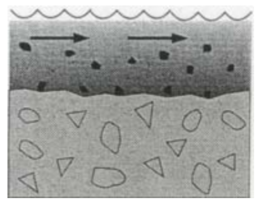
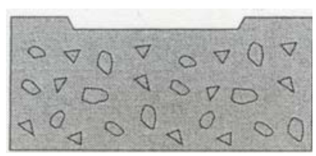
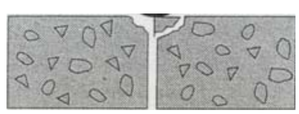
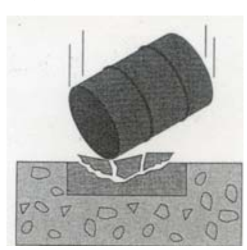
- **<u>Application conditions:</u>**

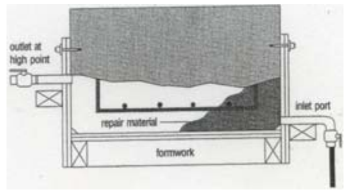
  Determining application conditions allows you to further determine the best repair material for the task at hand. Some of these application conditions include project time frame, weather conditions, access, and operating conditions. This will be discussed further in determining material properties.

Summary of application conditions is shown below [129].

Service Conditions: Structural Properties		
Performance Requirements	***Undesirable Response*** (results if wrong material is selected)	***Desirable properties***
Bond to substrate	Loss of bond, delamination; detachment of repair from substrate 	Tensile bond
Load carrying as intended by the engineer	Does not carry loads as anticipated, overstressing either substrate or repair material 	Modulus of elasticity similar to substrate
	Carries loads initially, but over time, the repair relaxes under creep deformation 	Very low compressive creep
	Drying shrinkage causes material to lose volume, reducing its ability to carry compressive loads. 	Very low drying shrinkage

Service Conditions: Exposure		
Performance Requirements	***Undesirable Response***	***Desirable properties***
Ambient Temperature change	Cracking in repair material due to thermal contraction stresses 	Thermal coefficient similar to that of substrate
	Spaling due to thermal expansion stress in substrate 	Thermal coefficient similar to that of substrate
Temperature changes within repair material at early ages	Deformation due to thermal expansion from high exotherm 	Low exotherm during curing
	Cracking due to thermal contraction stress in repair material 	
Atmospheric gases and chemical contact	Corrosion of reinforcing steel 	Low permeability, no cracks
Moisture conditions, saturation freezing and thawing	Disintegration of cement matrix 	Resistance to freezing and thawing
Moisture condition	Cracking due to drying shrinkage stresses 	Very law drying shrinkage; low permeability

Service Conditions: Dynamic Loading		
Performance Requirements	***Undesirable Response*** (results if wrong material is selected)	***Desirable properties***
High velocity flow	Erosion by cavitaion 	High compressive strength; high tensile strength; small maximum size aggregate
Low velocity flow with water borne debris	Erosion by abrasion 	High abrasion resistance; high compressive strength; large maximum aggregate size.
Vehicle wheels	Abrasion damage to surface 	High compressive strength; high abrasion resistance
	Edge spalling at joints 	High compressive, tensile, and bond strengths; tensile anchorage into substrate
	Delamination 	
Impact	Spalling 	High compressive, tensile, and impact strengths; Internal tensile reinforcement

Application Conditions		
Condition	***Performance Requirements***	***Desirable properties***
Constructability	Quick turn-around time	Rabid strength gain
	Flowability 	High flow
	Trowel overhead	Non-sag
Application Conditions		
Condition	***Undesirable Response*** (results if wrong material is selected)	***Desirable properties***
Appearance	Drying shrinkage cracks 	Low drying shrinkage
	Plastic shrinkage cracks 	Low surface water loss during placement

4.4.1 Determining Material Properties

Once the project objectives are determined the next step is to select a repair material that will allow for a successful repair given the previously determined conditions. This proper selection requires an understanding of how the repair material will respond under the expected conditions. Each separate condition could cause a response to occur in many spots in the repaired area. The surface, repair material, reinforcing steel, interface, or original concrete could all experience some sort of responses. These responses could eventually lead to a failure in any of the affected areas.

Knowing how the repair material will respond to the various conditions will make it possible to determine the material properties required for a proper repair. However, many times when one or two properties are optimized it will be at the expense of other required properties. Whenever numerous properties are needed for a successful repair those properties must be prioritized in order to know which ones are most critical for success. *A repair material should not be selected until properties that are needed for a successful repair are determined.* However, most of these properties are not provided by repair material manufacturers. The properties of major concern are:

(a) Bond Strength

Bond strength determines how well the repair material will bond to the existing concrete substrate. It is also the primary requirement in order to achieve a successful repair. In most cases if the substrate is properly prepared then there will be sufficient bond strength. Cases where there is a bond failure with a properly prepared substrate could be due to internal stresses developed when there is a difference in thermal properties or dimensional behaviors. These failures are not due to insufficient bond strengths.

(b) Dimensional Behavior

The difference in dimensional behavior between the new repair material and the existing concrete substrate is a key contributor to a failure in the repair. The primary dimensional properties are drying shrinkage, thermal coefficient of expansion, modulus of elasticity, and creep. When these properties in the substrate and repair material differ then there will be a difference in volume changes, which could affect appearance, durability, bond strength, and the ability for the repair area to carry loads.

i. Drying shrinkage

This property is of more concern when using cementitious repair materials. This is because cement based materials typically have larger volume changes during drying. Furthermore, many times more water is added than needed for hydration and as soon as the repair material adjusts to the humidity of the new environment the repair material will shrink in volume. This shrinkage volume change can cause many problems especially when dealing with an older substrate that has already obtained a stable shrinkage volume. This volume change of the repair material will cause new internal forces at the bond interface, which could lead to failure.

ii. Thermal Coefficient of Expansion

The coefficient of thermal expansion tells you how much a material will expand or contract with a temperature change. Since all repairs will be performed outside where there will always be a temperature change this property must be evaluated. When comparing the thermal coefficient of expansion of existing concrete with that of the repair material, cement-based materials typically have similar values. However, this is not the case with repair materials containing a polymer-matrix. They have a wide range of values that varies from 4 to 18 times greater than that of concrete. It is possible to decrease these values by adding some sort of filler or aggregate, but it will still yield a thermal coefficient of expansion 1.5 to 5 times greater than concrete. These differences in values can potentially cause internal stresses to develop around the bond interface. Because of polymer-matrix based repair materials have a larger coefficient of thermal expansion they are typically recommended for smaller repairs and not large repairs.

iii. Modulus of Elasticity

The modulus of elasticity measures how stiff a material is or in other terms how much a material will deform with a given load. The higher the modulus of elasticity the less it will deform under a load. When determining this property determine whether the repair will be for structural use or non-structural use. If it will be in a structural application that will be subjected to loads make sure the modulus of elasticity is the same or greater than that of the concrete substrate. If it is a non-structural application with no subjected load a lower modulus of elasticity material can be used to avoid cracking.

iv. Creep.

Creep is a property that will determine how the material will deform over time with a sustained load.

(c) Durability Properties

For concrete to be considered durable it must be able to withstand numerous service conditions, weathering, chemical attack, and abrasion. Many times failure or deterioration in concrete occurs because of the lack of durability in the concrete. This is why it is important to determine the cause of failure so that the durability problem can be corrected with a durable repair.

i) Permeability

ii) Water Vapor Transmission

Water vapor transmission is when water vapor flows through the existing concrete. The water vapor transmission rate is how quickly the water vapor is allowed to flow. This is a common occurrence with bridges that span over water and/or bridges in a humid environment. This water vapor transmission is very important in areas that experience numerous freeze-thaw cycles; because if an impermeable repair material is used it is possible that the water vapor can become trapped under the repair in the existing concrete substrate. Once the entrapped water vapor freezes, a hydraulic pressure builds up which could possibly cause a failure in the bond. It is also possible that the water vapor could gradually build up causing the concrete substrate to become critically saturated. This build up could very likely cause the substrate to experience freeze-thaw deterioration. For these reasons impermeable repair materials are not recommended for large repairs or thin patches.

iii) Freeze-Thaw Resistance

Freeze-thaw deterioration is typically defined as the failure of porous aggregate particles or the cement matrix when the material freezes while critically saturated. The failure occurs because of the expansive pressure exerted within the material when the water freezes. If a repair will be subjected to many freeze thaw cycles it is important that the repair material has good freeze-thaw resistance.

iv) Scaling Resistance

Scaling is another type deterioration that occurs because of freezing and thawing cycles. Scaling is when the surface portion of the concrete flakes off. This is again caused from hydraulic and osmotic pressures associated with freezing. If a repair will be subjected to freeze-thaw conditions then it is important that the repair material has good scaling resistance.

v) Sulfate Resistance

Sulfate attack is a form of deterioration that causes the decomposition of some of the binder compounds found in hydrated cement. The primary sources of naturally occurring sulfates come from decaying organic matter in marshes and shallow lakes. Structures such as bridge piers, bridge columns, and highway pavements subjected to this kind of environment must have good sulfate resistance to prevent deterioration.

vi) Alkali-Aggregate Reaction

An alkali-aggregate reaction occurs within the existing concrete. It is a chemical reaction that takes place between alkalis from the Portland cement and various parts of aggregate. The two most general types of this reaction are alkali-silica reaction and alkali carbonate reaction. Both types of reaction will cause excessive expansion and cracking of the existing concrete. When looking at repair materials, many manufactures will recommend adding some sort of coarse aggregate in repairs over 1 to 2 inches. Unlike aggregate selection for concrete, many times the selection of coarse aggregate for the repair material is overlooked. This overlooked choice of aggregate could cause a failed repair, because many cement based repair materials contain higher levels of alkalinity. Whenever coarse aggregate is going to be added to a repair material it is important to investigate the aggregate-repair material combination more thoroughly.

vii) Abrasion Resistance

Abrasion resistance is simply the ability of the concrete or repair material to resist being worn away from rubbing or friction. This resistance can be affected by factors such as compressive strength, mixture proportions, type of material, quality of aggregate, surface treatment, curing, and finishing procedures.

(d) Mechanical Properties

In order for a repair material to perform correctly it must have the proper mechanical properties to allow it to carry and transfer loads as concrete would. These loads could come in the form of external or internal. External loads are applied loads that could come from traffic or weather. Internal loads come from different dimensional changes in the concrete and repair material. Proper mechanical materials are vital for a successful repair, because it does matter how well the repair material bonds to the substrate if it can't withstand the service loads. Up to my knowledge, these are the only properties that are provided by the manufacturers.

i) Tensile Strength

The tensile strength of a material is simply how well that material can withstand tensile stress. Tensile strength of a material is important in repairs that will be subjected to some sort of tensile stress. In these cases the tensile strength must be examined. It also must be noted that the tensile strength of repair materials can vary significantly and must be checked.

ii) Flexural Strength

The flexural strength of a material is very important in bridge repairs, because it measures how much the material can resist bending. With that being said, the flexural strength should be considered in any repair that will be subjected to bending.

iii) Compressive Strength

The compressive strength is one of the most important mechanical properties of a repair material, because it measures how well the material can withstand compressive loads, which is most common on roads. If the compressive strength of the repair material differs too much from the concrete substrate it will cause excessive load transfers to the higher strength material. Therefore, it is typically recommended to pick a material with a similar compressive strength to that of the concrete substrate. However, many times increasing the cement content is used to increase the compressive strength, which in return could negatively affect other important properties.

(e) Constructability Properties

The constructability properties of a repair material are defined during the earlier stages of the material. These properties include curing time and plastic properties. In many cases repair materials are designed to make an easier repair job, but this ends up affecting other important properties. When selecting repair materials it is very important to make sure the other properties such as mechanical and durability are properly matched for your application, but if certain construction issues are not met it could cause undesirable repair results. These construction issues can be determined by limited workspace, traffic, required completion time, and many more. It doesn't matter how well a material will perform in place if it cannot be properly applied.

4.4.2 Materials

The choice of the repair material is directly related to the function of the repair and the expected service life of the structure after repair. Thus, repair could be performed cheaply to prolong the use of the structure for a limited life, or more expensively such that no remedial work will be required for many years.

The primary ingredients for most repair materials include one or more of the following:

(i) Ordinary or rapid hardening Portland cement

(ii) Epoxy resins

(iii) Polymer latex

(iv) Polyester resins

(v) Polyvinyl accelate

(vi) Fine and /or coarse aggregate filler

The use of Portland cement types I to V as the basic component in the repair material is always worth considering as it provides a comparable material to the concrete being repaired and is usually a less expensive alternative. The disadvantage associated with the use of cement-based repair material is basically the lower bond between old concrete and repair material. To increase this bond, various types of bonding agents are used either at a surface preparation stage or in the concrete repair mix. The incorporation of a suitable bonding emulsion such as polymer latex in cement mortar will improve its bonding to existing concrete, reduce its permeability and increase its tensile strength [8-9]. Polymer emulsion alone is effective for bonding fresh mortar to the surface of hardened concrete, provided it is not allowed to dry out before fresh mortar or concrete is placed. The materials most commonly referred to in this category are Styrene Butadiene Rubber (SBR) and acrylics. Another polymer bonding agent with a similar purpose is Polyvinyl acetate (PVA) which has the tendency of demulsifying on contact with moisture, thus its use is only advisable in situations where the concrete remains permanently dry.

Epoxy resin compounds are solvent free compounds cured by chemical reactions between the resin and the hardener [9, 125]. The mechanical properties of the epoxy may be altered through a variation of the three main components, resin, hardener, and the filler. The rate of curing heat generation during curing, viscosity, etc., may be altered by changing the ratios of the main components.

The majority of epoxies are supplied in pre-measured proportions, and the sizes of the packs are such that the proportions are correct if the entire contents of the packs are mixed in one batch. Splitting packs may cause added variation in the output product, thus it is usually discouraged. Epoxies need a thorough mixing usually using a mechanical stirrer, many failures occurred due to attempts of mixing by hand.

Epoxies tend to cure at a fast rate; this is both an advantage and a disadvantage. It is considered a disadvantage if it hardens prematurely prior to the end of its placement in the repair location or hardens on the equipment and tools being used in the repair. Some hardeners are toxic in varying degrees; therefore, precautions should be taken during handling of epoxy, polyester and polyurethane resin formulations [126, 127].

The epoxy curing process generates large amounts of heat, thus thermal stresses will be set as it cools and debonding to a certain degree may occur [9, 125]. The strength of many epoxy resin formulations under direct tension is high, but effective strength may be much lower when the joint tends to peel apart under load. The bonding between epoxy and old concrete is affected by the thermal history of the joint, and the joint temperature during the test. The bond is also affected by the moist or dry condition history of the joint, cycles of wet and dry conditions indicated a reduction in bond strength [128-130].

4.4.3 Selection of repair Materials

A variety of repair materials have been formulated to provide a wide range of properties. Since these properties will affect the performance of a repair, selecting the correct material for specific application requires careful study.

Concrete repair materials have been formulated to provide a wide range of properties. It is likely that more than one type of material will satisfy the design criteria for durable repair of specific structure. In these cases other factors must be taken into consideration which includes:

i Ease of application

ii Cost

iii Available labor skills and equipments

iv Shelf life of the material

v Pot life of the material.

A guideline for the selection of repair materials is shown in Figure 4.1. Typical characteristics of selected repair materials are shown in table 4.1 [129].

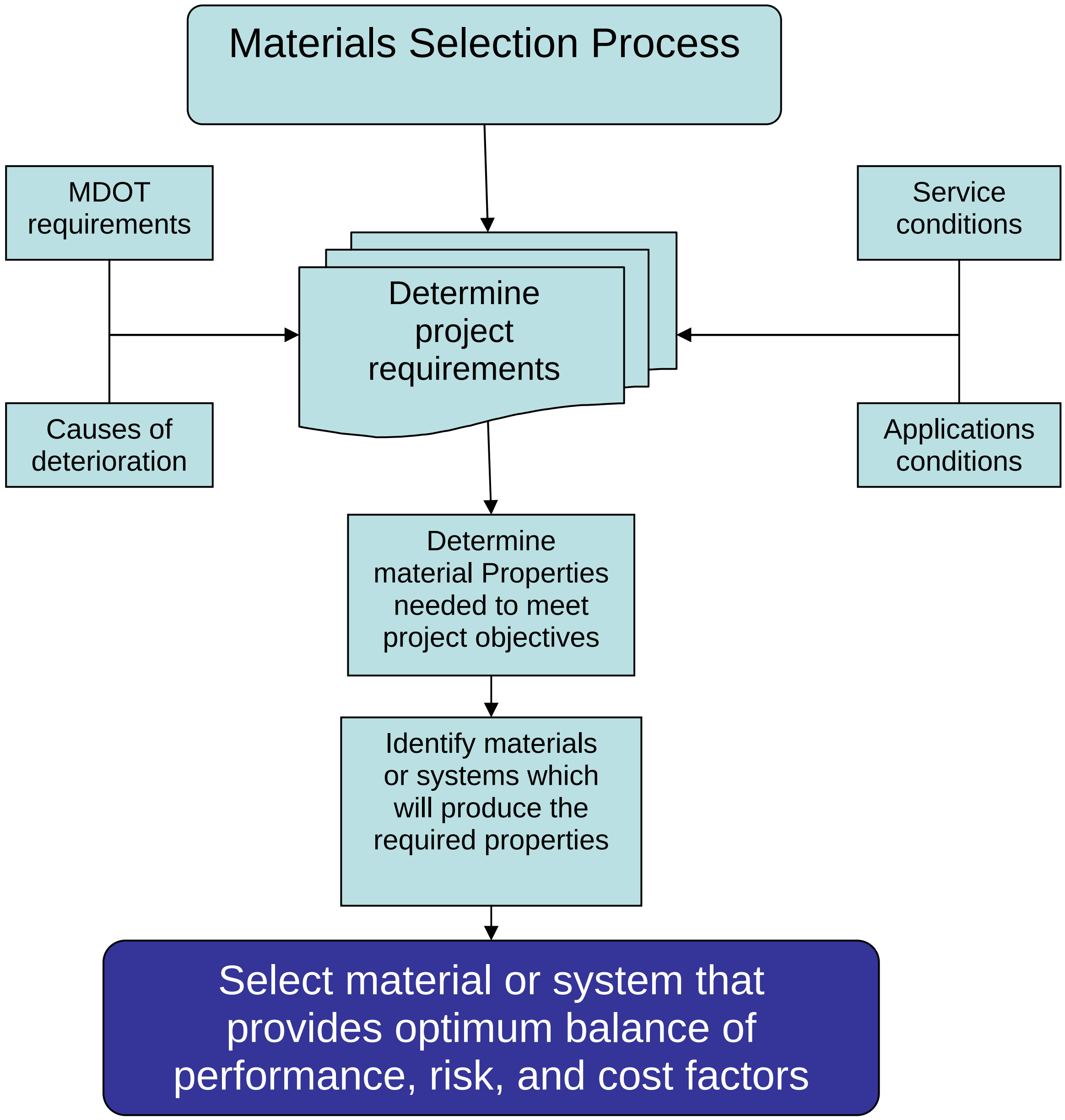


Figure 4.1 *Procedure for selection of repair materials [129].*

Table 4.1 Typical characteristics of selected repair materials [129]

Materials	Ingredients		Application Requirements			Material Properties										
	Binder	Typical additives/admixtures	Thickness Limitations (in)	Installation Temperature (F)	Curing	Drying Shrinkage	Coefficient of Thermal Expansion	Compressive Strength (psi)				Elastic Modulus (psi)	Permeability (% of Concrete)	Freeze-Thaw Resistance	Non-sag Quality	Exo-therm
								1 Hr.	24 Hrs.	3 Days	28 Days					
Portland Cement Mortar	Portland Cement	Water Reducing Air-Entraining	0.5-2.0	40-90	Wet 7 Days	Moderate	Similar to Substrate	0	650	2500	5000	3.4x10^6	90	Good	Mod-erate	Mod-erate
Portland Cement Concrete	Portland Cement	Water Reducing Air-Entraining	>1.75	40-90	Wet 7 Days	Low	Similar to Substrate	0	650	2500	5000	3.8x10^6	90	Good	N/A	Low
Microsilica-Modified Portland Cement Concrete	Portland Cement	Silica Fume, HRWR, Air-Entraining	>1.25	40-90	Wet 7 Days	Low	Similar to Substrate	0	3000	4000	7500	4x10^6	60	Good	Good	Low
Polymer-Modified Portland Cement Concrete	Portland Cement	Polymer Latex	>1.25	45-95	Wet 2 Days	Low	Similar to Substrate	0	2000	4000	6000	2.5x10^6	50	Excellent	N/A	Low
Polymer-Modified Portland Cement Mortar	Portland Cement	Non-Sag Fillers, Polymer Latex or Powder	0.25-2.0	45-95	Moist 3 Days	Moderate	Similar to Substrate	0	1500	3000	5000	2.5x10^6	50	Excellent	Low to Excel-lent	Mod-erate
Magnesium Phosphate Cement Concrete	Magn. Phosphate Cement		>0.50	0-100	Air	Low	Similar to Substrate	2000	6400	7000	84000	4.7x10^6	90	Good	Low	High
Preplaced Aggregate Concrete	Portland Cement	Pozzolans, Fluidifier	3.0	40-90	Wet 7 Days	Very Low	Similar to Substrate	0	500	2250	45000	3.8x10^6	100	Good	N/A	Low
Epoxy Mortar	Epoxy Resin	Sand	0.13-0.38	50-90	Air	Low	1.5-5x Concrete	0	9000	11000	12000	1.6x10^6	10	Excellent	Mod-erate	High
Methyl Methacrylate (MMA) Concrete	Acrylic Resin		0.25-0.5	20-120	Air	Moderate	1.5-5x Concrete	4000	12000	12000	12000	2.0x10^6	10	Excellent	N/A	High
Shotcrete	Portland Cement	Silica Fume, Pozzolans, water reducing accelerator latex	>0.5	40-90	Wet 7 Days	Moderate	Similar to Substrate	0	800	3500	5000	3.8x10^6	60	Good	N/A	Low

4.5 Case studies of failures in repair material or repair techniques used by MDOT in Mississippi

1- *Spalling of repair material from concrete*

Figure 4.2 *A case study of failure in repair material due to the size of repaired area.*

Figure 4.2 represents a case of repair that was carried out in the Hattiesburg area. The pavement was repaired using Pot-Fill repair material. After the repair, part of the repair material delaminated and spalling of repair material was observed. In this case the following mistakes were made.

- An area of the pavement was **cut** to an approximate size of 3 by 6 ft. This means an enlargement of the repaired area which is not recommended. Because, the larger the repaired area the higher the associated shrinkage of repair material will be. Thus, the higher the chance of repair material delamination from parent concrete.
- Using polymer based materials is associated with an exotherm process which sets internal stresses at the repair material- concrete interface. This problem will be magnified when using large quantities of repair materials which is the case in this situation.
- The repaired area is relatively large. Thus it is recommended to use expansion joints in this case.
- The parent concrete surface was not prepared properly to reach a sound clean layer of concrete as was discussed earlier in this chapter.
- By examining the second picture, it is clear that the repair material bonded well to the side of the repaired section. However, it seems that under heavy loading the middle of the repaired section deformed significantly causing cracking shown in the picture. This is similar to a case of a beam with both ends fixed. This could be due to a very low compressive creep of used repair material (see page 90).

Recommendations

1- Don't cut an area of concrete to prepare for repairing

2- If there is a need to repair a large area, then one must use expansion joints

Figure 4.3 *Spalling of repair material from parent concrete*

This is a common problem that was observed in many repaired bridges in Mississippi. By examining the failed section it is clear that the repair material bonded extremely well to the parent concrete. By looking at the above picture all one can see is parent concrete (no repair material was left behind). This is a common problem associated with not cleaning and preparing the concrete surface properly before applying the repair material. Thus the problem is not the selection of the right repair material, but rather surface preparation (See section 4.1).

Figure 4.4 *A case study showing improper use of repair technique*

This is a case study that was examined in District III. By examining, one can observe the discoloration of bridge deck due to underlying corrosion. Thus patching repair material at the surface will not solve the problem. Proper cleaning of reinforcement or at least sealing of corrosion cracks is recommended in this case.

Figure 4.5 *Abrasion of repair materials*

Abrasion of repair materials is another common problem that was observed in many districts. Before selection of any repair material, abrasion resistance of the material needs to be provided by repair materials manufacturers.

Figure 4.6 Cracking of repair materials

In this case a very thin layer of patching material was used to repair a concrete bridge deck which was deteriorated by abrasion. Apparently the repair material itself was damaged as cracks are noticed everywhere. This could be due to the use of repair material with a low tensile strength.

Figure 4.7 *Repair of joint failure due to continuous dynamic impact of traffic*

This is another common problem in which the repair material concrete interface is damaged due to the continuous impact of traffic.

CHAPTER 5

DETERIORATION OF BOND BETWEEN REPAIR MATERIAL AND CONCRETE DUE TO THERMAL INCOMPATIBILITY

5.1 Introduction

When repair is carried out by reinstatement of the section after removing the deteriorated concrete, the compatibility between the repair materials and the parent concrete becomes a major general concern. The concern is, however greatly magnified in climatic conditions where large fluctuations of temperatures, and thus of thermal expansions of repair material and parent concrete, would cause differential thermal strains at the repair material - concrete interface resulting in a possible damage to the bond or adhesion at the repair joint.

Epoxy resins are very commonly used repair materials. However, epoxies present significant problems with respect to the deterioration of bond at the repair - concrete interface due to thermal and mechanical incompatibility problem between epoxy and the parent concrete. The rapid curing characteristics of epoxy resins may be advantageous provided that the repair material remains workable for enough time. However, this rapid curing is associated with high heat generation. If the material hardens while hot, this will result in thermal stresses at the epoxy - parent concrete surface. This thermal stress is a result of the difference in thermal coefficient of expansion between the concrete and the epoxy resin. The coefficient of expansion of the epoxy is much larger than that of concrete. If the thermal stress exceeds the adhesion stress capacity, debonding may occur. However, the low elastic modulus of the epoxy resin may reduce the effect of the thermal stress. The strength of many epoxy resin formulations under direct tension is high, but the effective tensile strength of epoxy resin repairs may be much lower as the epoxy - parent concrete joints tend to peel apart under load.

When a composite material (concrete-repair material composite) is subjected to a temperature change, thermal stresses are created due to a mismatch in thermal expansion coefficients (CTE). The difference in the thermal coefficient of expansion between concrete and epoxy formulations can be altered by controlling the amount of aggregate to binder ratio, where the filler/epoxy ratio was varied in an attempt to vary (CTE) of repair materials. This considerable difference in coefficient of thermal expansion between epoxies and Portland cement does require careful consideration.

In terms of evaluating the performance of repair materials for bond between parent concrete and repair under cyclic heating and cooling, Morris Schupach [130] states in his paper Divorces and Ruptured Relations Between Epoxies and Concrete that a single high thermal shock sometimes can degrade the composite, as can any of the various cyclic changes over a period of time. He said that one does not have to make any calculations to see that the change in volume or shape of an epoxy due to changes in temperature, wetting and drying, freezing and thawing, or loads is likely to be very different from that of concrete to which it is attached. These differences can cause high stress at the bond line that may lead to failure.

In order to study the performance of repair materials under an aggressive environment, an extreme summer day in the State of Mississippi was simulated. The ambient temperature in Mississippi during summer months reaches 100^{o}F and the effect of intense direct solar radiation on a still day raises the temperature of concrete surfaces as high as 120^{o}F. During the night, the temperature of concrete surfaces goes down to as low as 50°F-60°F [131]. The daily temperature variation in summer can cause significant thermal strains at the repair-parent concrete expansions of the repair material and the parent concrete. This thermal incompatibility at the repair interface may significantly damage the quality of adhesion/bond between the repair and the parent concrete.

5.2 EXPERIMENTAL PROGRAM

Slant shear tests were carried out in accordance with British Standard (BS) 6319: Part 4, to quantitatively evaluate the bond between concrete and repair material.

Alternatively, we could have used ASTM C 882-91. However, using BS 6319: part 4, it is possible to simulate more realistic rough surface.

5.2.1 Slant Shear Test (B. S. 6319 No. 4: 1984)

Most uses of repair materials and resin compositions involve contact and adhesion to hydraulic cement concrete and require the development of a strong adhesion bond between these two materials. The purpose of this test is to evaluate the bond strength between two materials.

If the contact surface between two materials is subjected to a loading in compression, little information will be obtained since the only likely failure to occur will be due to compression failure of the weaker material, prior to failure of the bonded surface which is subjected indirectly to shear stresses due to the difference in elastic response of the two materials. Direct tension across the bonded surface will give a misleading value since the failure will be due to the tension failure of the weaker hydraulic cement. Therefore, the “pull - off “test will reflect the tensile strength of concrete rather than the bond between the repair material and the hydraulic cement.

The generation of pure shear stress at the bond surface between hydraulic cement and repair material or resin composition required an elaborate setup. A simpler approach is to apply a compressive load to a specimen taking the form of a composite prism with a bond surface running diagonally through it. This method is used to investigate the strength of an adhesive and is known as a diagonal slant shear bond test. This test method subjects the bond surface to a combination of shear and compressive stresses, the type of regime most likely to be encountered in concrete structures.

The ratio of shear to compressive stress increases as the angle between the bond surface and the vertical axis is reduced. An angle of 30 degrees has been found to be the shallowest practicable angle at which a joint can be made in a prism of modest dimensions. A test prism may be made of two halves made of hydraulic cement concrete with the resin composition forming a scarf joint between the two halves. Or half the test prism may be made of hydraulic cement concrete and other half made of resin mortar or resin concrete, the interface forming the scarf joint.

The bond surface may be modified to simulate the variety of applications and circumstances for which resin compositions are used.

When the purpose of the testing is to provide basic data with which to compare the performance of resin compositions and / or repairing materials from different sources, it is recommended that the plaques are made from a high - strength concrete mix. It is important that the compressive strength of the concrete mix used is quoted against the slant shear bond strength of the resin composition or repairing material.

5.2.2 Preparation of Test Specimens

Fifteen aluminium molds were designed so that from each mold two specimens having a dimension of 150 x 150 x 55 mm (6 x 6 x 2 in.) may be cast. Additionally, 3 x 6 in. control cylinders were also made to monitor the mix strength. The plaques and cylinders were demolded after 24 hours. Then, they were cured at room temperature. The plaques were made from a high strength concrete mix using the following materials and mix compositions:

		Yield, *cu ft*
Ordinary Type I Portland Cement (8.00 SK), *lb*	752	3.83
Concrete Sand, *lb*	1365	8.32
#67 Limestone, *lb*	1733	10.98
Water, *lb (gal-US)*	225 (27.0)	3.61
Air entrapped, %	1.5% +/- 1.0 %	0.41
	TOTAL	**27.15**
GRACE WRDA 35 water reducer, *oz-US*	22.6	
GRACE ADVA-flow super plasticizer, *oz-US*	45.1	
Water / Cement ratio	0.3	
Slump, *in*	6.00 +/-1.00	
Concrete unit weight, *pcf*	150.1	
Average moisture corrections for coarse aggregate	1%	
Average moisture corrections for fine aggregate	5%	

The control cylinders broke at 7,183 psi after 146 days.

The test program required the splitting of the plaques in two halves along a 30^{o}angle. Before splitting the plaques, the plaques were grooved by a cutting wheel to a depth of about 5mm to ensure a perfect half split at the required angle of the plaque (Figure 5.1). Concrete plaques were then assembled with a trapezoidal steel plate and an elastomeric pad. A steel rod was located on the top of the plaque to help in guiding and promoting the crack along the desired angle. Compression loading was then applied slowly at a constant rate until the plaque was fractured (Figure 5.2). This procedure will assure a natural rough surface of crack. The fractured plaque is now ready for repair (Figure 5.3).

Figure 5.1 *A groove was made on every concrete plaque at an angle of 30 degrees.*

Figure 5.2 *Concrete Compression machine was used to split concrete plaques.*

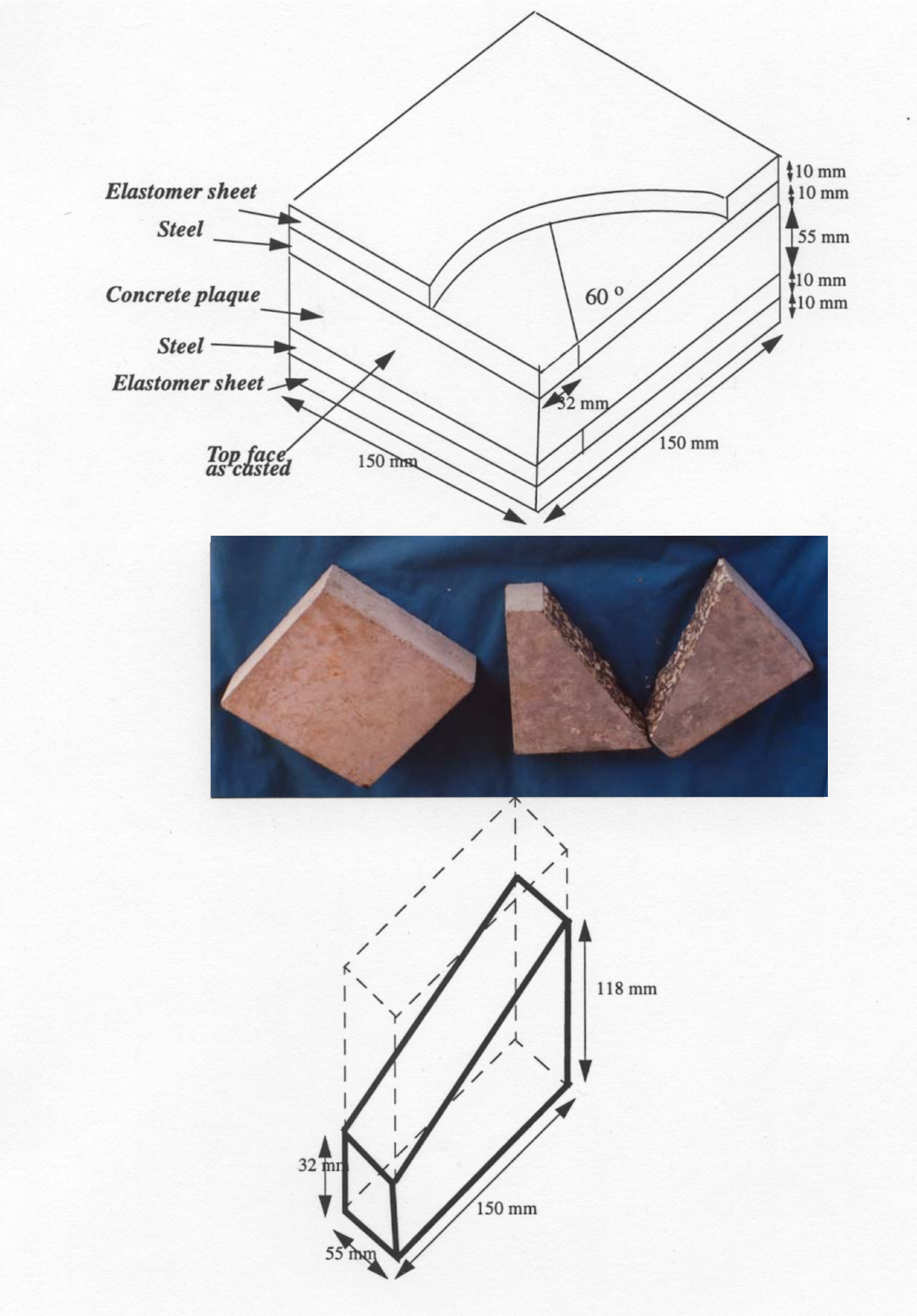


Figure 5.3 *Assembly for splitting concrete plaques at controlled angle*

5.2.3 Repairing of concrete plaques using Reinstatement of section method

This method is used to replace the deteriorated part of the structure with a new section of better properties with an aim of maintaining serviceability of the structure.

5.2.3.1 Materials Used for Repair

In the research program, the following commercially available materials were used for repairing:

#	Repair Material	Classification	Components and properties	Pre Treatment	Usage
1	Road Patch	It's a very rapid hardening Portland cement based horizontal repair mortar for load bearing substrates	one component mortar Mortar : Water 2.36L of water for 50lb bag	Precondition to (70+/-5)F	- Used in cold and hot climates - Used for repairing deteriorated concrete quickly
2	14K HY Flow	It's a one component, cementitious, non-metallic shrinkage compensated natural aggregate precision grout	one component mortar Mortar : Water For 55lb bag 2.36L- minimum flow 3.3L- moderate flow 4.5L-high flow	Precondition to (70+/-5)F	-Improves Effective Bearing Area - Doesn't deform under constant loads -Easy to place from non sag to flowable consistencies
3	Set 45	It's a one component magnesium phosphate based patching and repair mortar	one component mortar Mortar : Water Around 2litres of water for 50lb bag	Precondition to 70F for 24hrs before mixing	-Rapidly returns to service -Used mostly for permanent repairs -Bonds to concrete and masonry without bonding agent
4	Emaco T415	It is one component high performance product	one component mortar Mortar : Water Around 3.16litres of water for 55lb	Precondition to 70F for 24hrs before mixing	- Can be used in almost all environments -Reduces dependency on weather
5	HP LV	It's a epoxy resin and hardener system where low viscosity material is required	1.Epoxy Resin 2.Epoxy Hardener 3.Filler R:H:F 2:1:2	Epoxy Primer	-Used to seal non moving cracks in concrete -Used to set anchor bolts in drilled holes
6	HP Binder	It's a two component 100% solids epoxy resin system used as binder for making epoxy mortar	1.Epoxy Resin 2.Epoxy Hardener 3.Filler R:H:F 2:1:2	Epoxy Primer	-Used for spall repairs, setting anchor bolts -Used in applications where high quality, non shrinking epoxy mortar is required

7	HP GPA	It's a solvent free 2 to 1 epoxy resin and hardener system	1.Epoxy Resin 2.Epoxy Hardener 3.Filler R:H:F 2:1:2	Pre conditioning is done by sand blasting	-Used for bonding new concrete to old concrete -Used for grouting anchor bolts -Used for bonding of materials such as wood, metals etc.
8	Pot Fill	It's a repair system for concrete pavements that uses radically structured polymeric repair compound	Part A: Epoxy Resin Part B: Proprietary Accelerator Part C: Mineral Aggregates	Epoxy Primer	-Used for Rapid Repairing -Used for deep fill repairs to weather exposed concrete pavements (highways etc.)
9	Sika 2500 With Latex	Sika Latex R is an acrylic polymer latex	Part A: Sika Latex Part B: Cement paste material 1 gal of Part A for 50lb bag	Precondition material to (60-75)F	-Used in patching and flash coats -Used as bonding grout when mixed with sand and Portland cement
10	Sika 2500 with water	It's a one component very rapid hardening material for concrete	one component mortar Mortar : Water Approximately 5-5.5 pints of water for 50lb bag	Precondition material to (65-75)F	-Used in highway overlays and repairs -Used in full depth patching repairs
11	Sika Top 123 Plus	It's a two component polymer modified Portland cement, fast setting, non sag mortar	Part A: 1Gal of Latex Part B: 44lb bag of cement paste material	Precondition material to (65-75)F	-Used on vertical and overhead surfaces -can be used as a structural material
12.	RS 2 Sum	It is a styrene diluted unsaturated polyester-based polymer concrete	Part A: I Gal of resin Part B: 3 oz of hardener (2% of the resin) Part C: ½ cu. Ft of filler	Epoxy primer	- Used for general purpose repair of concrete structures
13	RS 2 Win	It is a styrene diluted unsaturated polyester-based polymer concrete	Part A: I Gal of resin Part B: 3 oz of hardener (2% of the resin) Part C: ½ cu. Ft of filler	Epoxy primer	- Used for general purpose repair of concrete structures

5.2.4 Pre - treatment

Before repairing, the surface of each fractured plaque was brushed to remove any loose material. The surface was then washed with clean water to remove any contaminating dust, and allowed to dry. The dry clean surface was primed prior to the application of repair material according to the manufacturer recommendation.

5.2.5 Repairing of the Split Plaque

The trapezoidal half - plaques were placed at the base of the 150 x 150 x 55 mm (5.906 x 5.906 x 2.165 in.) aluminium molds (Figure 5.4). After mixing the repairing material thoroughly using a hand mixer in a drill, each empty half of the molds was filled with the repairing material in 3 layers to prepare a full size plaque (Figure 5.5). The repaired plaques were then left to cure for the period recommended by the manufacturer.

Figure 5.4 *Trapizoidal half plaques placed in the mold.*

Figure 5.5 *Typical repaired concrete plaque.*

5.2.6 Preparation of Test Specimens from Composite Plaques for the Slant Shear Bond

After the recommended curing period, each plaque was sawn into three segments in accordance with BS 6319: No. 4: 1994 as shown in Fig. 5.6.

The sawn prisms of 55 x 55 x 150 mm (5.906 x 5.906 x 2.165 in.) are the repair material - concrete composite specimens to be used for the slant shear test (Figures 5.7 and 5.8).

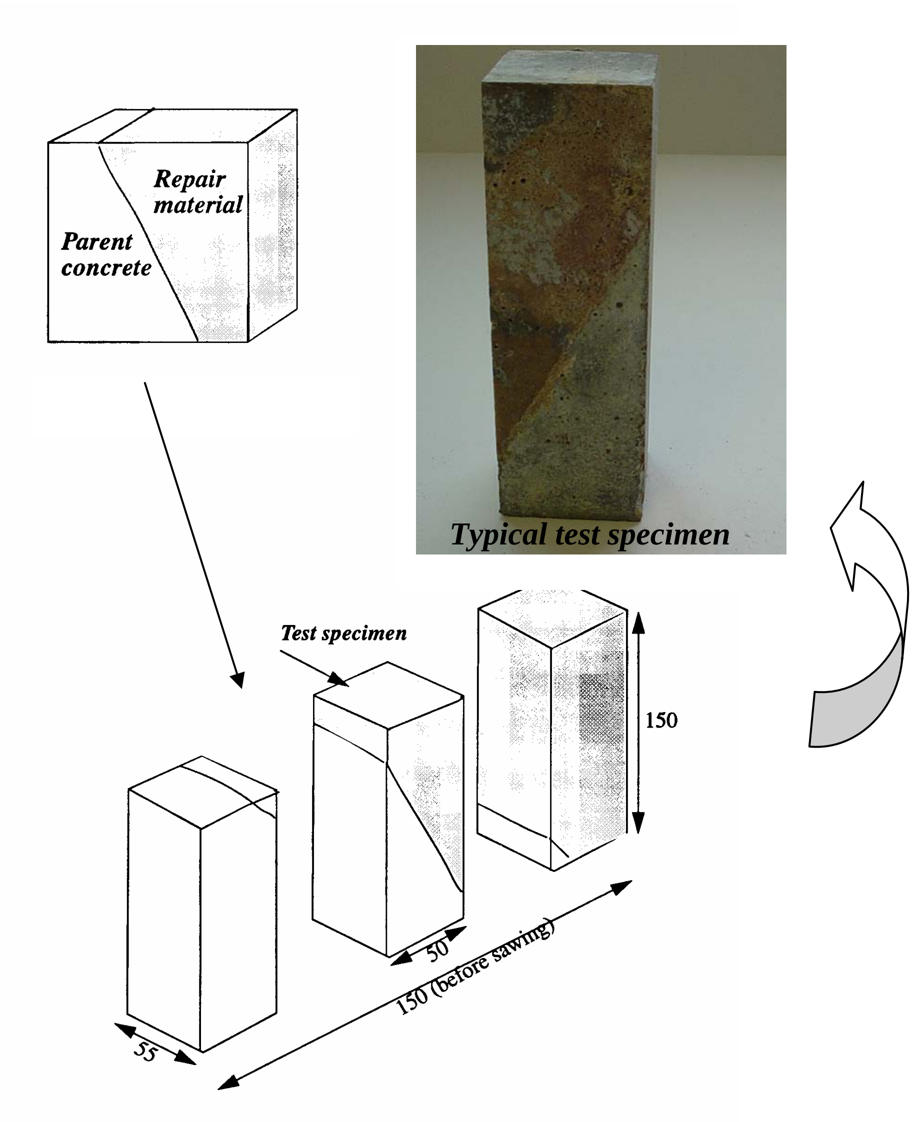


Figure 5.6 *Preparation of test specimen*

Figure 5.7 *Typical test specimen from resin based repair materials (Pot Fill, HP LV, HP Binder, HP GPA, Resurf II Summer, and Resurf II Winter).*

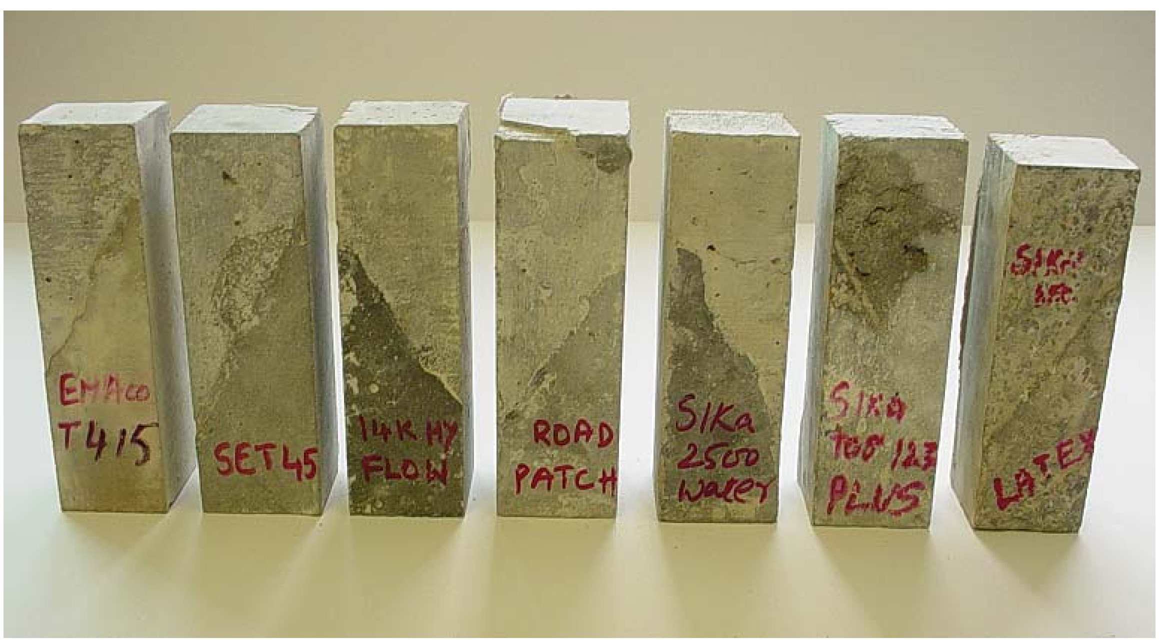


Figure 5.8 *Typical test specimens from cement-based repair materials (Emaco T415, Set 45, 14K HY Flow, Road Patch, Sika 2500 mixed in water, Sika 100 Plus, Sika 2500 mixed with Latex)*

5.2.7 Testing of the Repaired Samples

The sawn prisms of the repaired plaques were then placed in an automatically programmed temperature controlled oven which executed a thermal cycle 65^o - 120^o - 65^o within the 24 hours with a constant humidity of 80% ± 10% thereby simulating an extreme summer day temperature in Mississippi (Figure 5.9).

The following characteristics of the thermal cycles were used:

- The chamber temperature was lowered from room temperature to 65^oF in a period of 5 minutes.
- The temperature was held constant for 4 hours at 65^oF.
- The temperature was raised to 120^oF in a period of 5 hours at a constant rate of 11^oF / hour.
- The temperature was then constant for 7 hours at 120^oF.
- The temperature is then dropped to 65 in 5 hours at a rate of 11^oF / hour.
- The temperature is then kept constant at 65^oF for 3 hours.

The samples were placed in the oven for 60 cycles and then tested under compression. A similar set of control specimens which was not subjected to heating - cooling cycles were left at constant laboratory room temperature for the same age. All specimens were capped before being tested in compression (Figures 5.10 and 5.11).

Figure 5.9 *Oven with specimens inside*

Figure 5.10 *Capped control specimens*

Figure 5.11 *Capped aged specimens*.

5.3 Criterion Used for Interpretation of Evaluation Test Results

The following criterion was used for the interpretation of slant shear evaluation tests carried out on crack - repaired prisms:

(i) **Diagonal failure at the joint at a significantly lower load than the control, with no concrete failure.** This indicates inadequate bond and an unsuccessful repair.

(ii) **Diagonal failure at the joint at a load only a little lower than the control with little or no concrete failure.** This indicates fairly good bond between repair material and parent concrete. The joint failure is most likely promoted by the additional stress caused at the joint due to the enhanced strain response of the lower - modulus repair material spread along the joint. This indicates an acceptable bond in some applications although the repair is not 100 percent successful.

(iii) **Diagonal failure in the concrete parallel to the joint but about 5 mm away from it at a similar load to the control**. This indicates excellent bond between repair material and parent concrete and a successful repair.

(iv) **“Double - pyramid” failure of the same type as occurs in control specimens; the failure load may be equal to or superior to that of the controls.** This indicates a most successful repair in which monolithic performance has been achieved between repair material and parent concrete.

Slant shear tests were carried out, on repaired test specimens, using two environmental conditions for each repair method:

(i) Condition corresponding to uniform room temperature at about 20 C.

(ii) Condition corresponding to typical 24 hour fluctuations of temperature in typical summer days of Mississippi.

5.4 RESULTS AND DISCUSIONS.

5.4.1 Evaluation of the Quality of Adhesion and Bond between Repair Material and Concrete at room Temperature Behavior

A significant volume of concrete repair may be carried out on standard components which are indoors in a relatively protective environment, especially from the standpoint of temperature fluctuations. In order to evaluate the performance of repair materials under these mild static climatic conditions, slant shear bond strength tests have been carried out at static room temperature on composite prisms, repaired with a range of potential resinous and cementitious materials. The results of the slant shear bond strength at static temperature are given in Table 5.1. Mode of failure was examined according to the failure criterion mentioned above for all specimens (Figure 5.12-5.24).

Table 5.1 Results of Slant Shear Test for Control Specimens

		Slant Shear of Control Specimen		
	Material	Average *(psi)*	Stdev.	COV %
Cement –Based Materials	Road Patch	6056.6	375	6.2
	14K HY Flow	6074.8	1218	20
	Set 45	3498.9	188	5
	Emaco T 415	4189.6	944	22
	Sika 2500 with Water	6442.1	2418	37
	Sika 2500 with Latex	4436.1	1601	36
	Sika Top 123 Plus	5428.6	775	14
	Pot Fill	6401	1149	18
Resin –Based	HP Binder	3723.2	1663	45
	HP GPA	4229.1	546	13
	HP LV	4713.7	1261	27
	Resurf II S	6376.3	1518	23
	Resurf II W	7925.4	2450	31

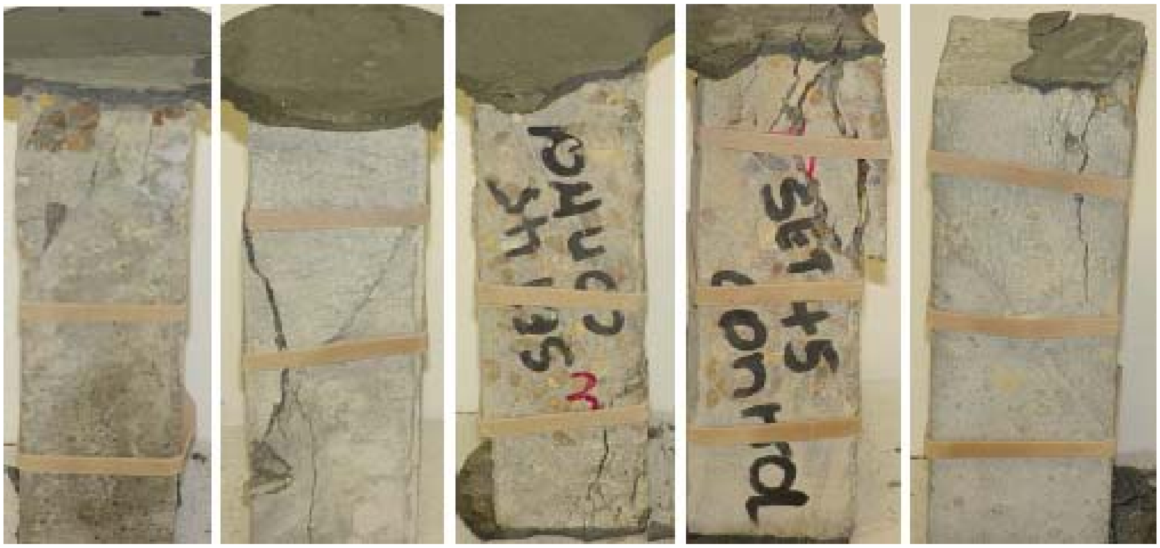
- Control concrete specimens broke at an average strength of 7,183 psi after 146 days (same age as repaired specimens).

Sample1 MOF(iii) Sample2 MOF(iii) Sample3 MOF(iii) Sample4 MOF(iv) Sample5 MOF(iv)

Figure 5.12 *Mode of failure of concrete plaques repaired using Road Patch.*

Sample1 MOF(iv) Sample2 MOF(iv) Sample3 MOF(iv) Sample4 MOF(iv) Sample5 MOF(iv)

Figure 5.13 *Mode of failure of concrete plaques repaired using 14K HY Flow.*

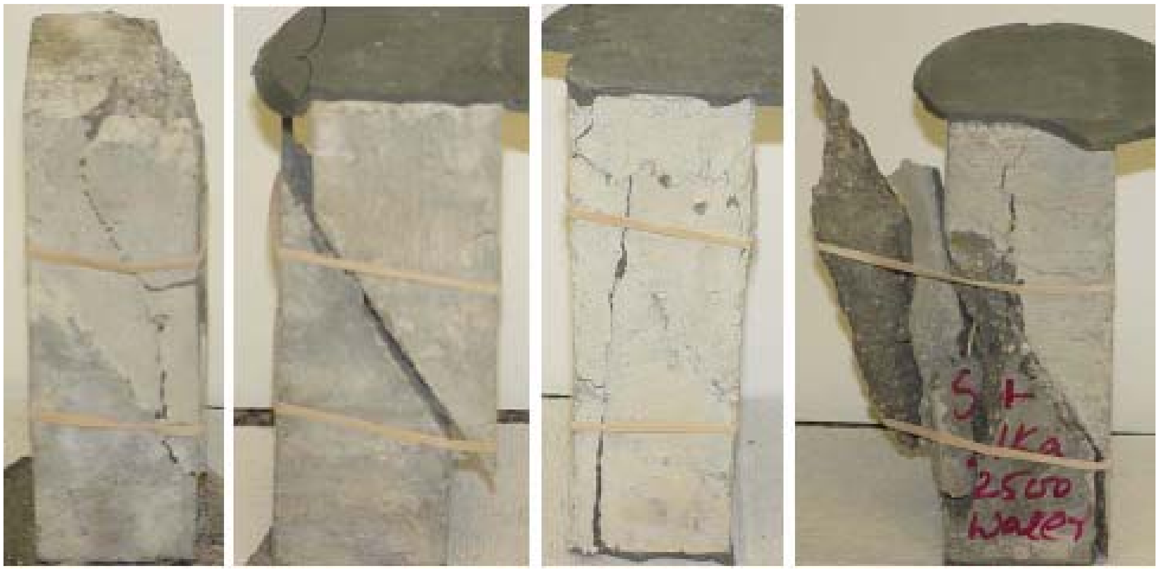


Sample1 MOF(i) | Sample2 MOF(i) | Sample3 MOF(i) | Sample4 MOF(i) | Sample5 MOF(ii)

Figure 5.14 *Mode of failure of concrete plaques repaired using Set 45.*

Sample1 MOF(i) | Sample2 MOF(i) | Sample3 MOF(i) | Sample4 MOF(i) | Sample5 MOF(ii)

Figure 5.15 *Mode of failure of concrete plaques repaired using Emaco T415.*



Sample1 MOF(iv) | Sample2 MOF(iii) | Sample3 MOF(iv) | Sample4 MOF(iii)

Figure 5.16 *Mode of failure of concrete plaques repaired using Sika 2500 mixed with water.*

Sample1 MOF(ii) | Sample2 MOF(ii) | Sample3 MOF(i) | Sample4 MOF(ii)

Figure 5.17 *Mode of failure of concrete plaques repaired using Sika 2500 mixed with Latex.*

Sample1 MOF(iv) | Sample2 MOF(iii) | Sample3 MOF(iii) | Sample4 MOF(iii) | Sample5 MOF(iv)

Figure 5.18 *Mode of failure of concrete plaques repaired using Sika Top123 Plus.*

Sample1 MOF(iii) | Sample2 MOF(iii) | Sample3 MOF(iii) | Sample4 MOF(iii) | Sample5 MOF(iv)

Figure 5.19 *Mode of failure of concrete plaques repaired using Pot Fill.*

Sample1 MOF(ii) | Sample2 MOF(ii) | Sample3 MOF(ii) | Sample4 MOF(iii) | Sample5 MOF(ii)

Figure 5.20 *Mode of failure of concrete plaques repaired using HP Binder.*

Sample1 MOF(ii) | Sample2 MOF(iii) | Sample3 MOF(ii) | Sample4 MOF(ii) | Sample5 MOF(ii)

Figure 5.21 *Mode of failure of concrete plaques repaired using HP GPA.*

Sample1 MOF(ii) | Sample2 MOF(ii) | Sample3 MOF(ii) | Sample4 MOF(iii) | Sample5 MOF(iii)

Figure 5.22 *Mode of failure of concrete plaques repaired using HP LV.*

Sample1 MOF(iv) | Sample2 MOF(iii) | Sample3 MOF(iv) | Sample4 MOF(iv) | Sample5 MOF(iv)

Figure 5.23 *Mode of failure of concrete plaques repaired using Resurf II (summer cured).*

Sample1 MOF(iii) | Sample2 MOF(iv) | Sample3 MOF(iii) | Sample4 MOF(iv) | Sample5 MOF(iii)

Figure 5.24 *Mode of failure of concrete plaques repaired using Resurf II (winter cured).*

5.4.1.1 Resinous Repair Materials:

It is seen from the slant shear bond strength data of Table 5.1 that for the three resinous materials Pot Fill and Resurf II (summer cured) , the composite prisms failed at loads corresponding to average slant shear bond strengths which were 11 percent lower than the strength of the concrete control. For the case of composite prisms repaired with Resurf II (winter cured) the composite prisms failed at loads corresponding to average slant shear bond strengths which were 10 percent higher than the strength of the concrete control. On the other hand, for the HP resins (HP Binder, GPA and LV), the repaired prisms failed at much lower load which corresponds to reduction in strengths of 48, 41 and 34 % in comparison to control concrete specimens. On an average, the resins showed 22 percent lower strength for the repair in slant shear bond than the concrete control.

The typical failure mode for the resin repaired composite prisms using Pot Fill, Resurf II (summer cured) and Resurf II (winter cured) are characterized by modes iii and

iv. Primarily they were characterized by the crushing of concrete in the lower portion of the specimens with a diagonal failure along the repair joint (Fig 5.19, 23 and 24).

The performance and behavior of the repair resins HP Binder, GP and LV were, however, distinctly different from the performance of the other three resins. The composite prisms repaired with HP resins failed at a load corresponding to average slant shear bond strength which was 41 percent lower than the compressive strength of the concrete control as compared to an average of about 4 percent lower than control slant shear bond strength shown by the other three resins (Pot Fill and Resurf II). Furthermore, the mode of failure was characterized in most cases by diagonal failure along the repair joint rather than failure by the crushing of concrete observed for the three resins.

5.4.1.2 Cementitious Repair Materials:

The slant shear bond strength data (Table 5.1) for the cementitious repair materials Set 45, Emaco T415, Sika 2500 with Latex and Sika Top 123 Plus show that for the four cementitious repair materials, the failure occurs at loads corresponding to slant shear bond strengths which are 31 to 55 percent lower than the control concrete compressive strength. The worst performance corresponding to 55 percent lower than control strength is shown by Set 45 which is one of the most used materials by MDOT maintenance personnel. On the other hand, Road Patch, 14K HY Flow and Sika 2500 with water, show comparable performance to good resinous materials (i.e. Pot Fill and Resurf II) with a reduction in slant shear strength of 17-22% only. Sika 2500 mixed with water showed the best performance with a slant shear strength reduction of only 17% compared to parent concrete.

The typical failure mode for Set 45, Emaco T415, Sika 2500 (Latex) and Sika Top 123 is characterized by a disturbed joint failure without any signs of concrete crushing. The very low w/c ratio for Road Patch, 14K HY Flow and Sika 2500 (Latex) exhibits repair to parent concrete bond performance comparable to the resinous repair materials Pot Fill, Resurf II (S), and Resurf II (W).

5.4.1.3 Discussion:

Three of the six resinous materials (Pot Fill, Resurf II (S) and Resurf II (W))used for repair show excellent bond between repair and concrete by failing at loads corresponding to slant shear bond strength on an average only 3 percent lower than the compressive strength of the control concrete (ASTM C109). The failure is characterized by the crushing of concrete with no diagonal failure along the repair joint. The failure load in conjunction with the characteristic failure mode shows an excellent and completely successful repair.

However, the other three resinous materials (HP Binder, GPA and LV), failing along the diagonal repair joint at a slant shear bond strength lower than control concrete strength indicate inadequate bond between repair material and parent concrete and an unsuccessful repair.

Three of the cementitious repair materials fall along the diagonal repair joint at a slant shear bond strength much lower than control concrete strength. Repair with Sika 2500 (Latex) shows the best performance among all used cement-based materials as it fails at a shear bond strength only 17 percent lower than the control strength. In many cases, such repair should be acceptable. However, repair by some widely used commercial construction materials such as Set 45 shows markedly weak bond between repair and the parent concrete as it fails along the repair joint at a slant shear bond strength 55 percent lower than the control strength. Such a repair is indicated to be unsuccessful and raises valid questions about the continued use of such materials.

5.4.2 Evaluation of the Quality of Adhesion and Bond between Repair Material and Concrete of Repaired Specimens Subjected to Cyclic Variation of Temperature

In order to evaluate the effect of thermal fluctuations, an investigation has been carried out into the deterioration of bond at repair-concrete interface as a result of thermal cycling simulating the daily fluctuations of temperature in extreme summer months in Mississippi. The results are tabulated in Table 5.2 and Figure 5.25. Failure modes are shown in Figures 5.26-5.38.

Table 5.2 *Results of Slant Shear Test for Test Specimens Subjected to Thermal cycling (60 cycles).*

		Slant Shear of Control Specimen			Slant Shear pf Aged Specimen			%
	Material	Average *(psi)*	Stdev.	COV %	Average *(psi)*	Stdev.	COV	Reduction in Slant Shear Strength
Cement –Based Materials	Road Patch	6056.6	375	6.2	3632.5	964	27	40.0
	14K HY Flow	6074.8	1218	20	4777.1	691	14	21.3
	Set 45	3498.9	188	5	4178.1	546	13	-19.41
	Emaco T 415	4189.6	944	22	4139.4	1049	25	1.2
	Sika 2500 with Water	6442.1	2418	37	5203.4	1579	30	19.2
	Sika 2500 with Latex	4436.1	1601	36	3520	773	22	20.7
	Sika Top 123 Plus	5428.6	775	14	4486.9	485	11	17.4
Resin –Based Materials	Pot Fill	6401	1149	18	8134.1	1154	14	-27.1
	HP Binder	3723.2	1663	45	2800.13	445	16	24.8
	HP GPA	4229.1	546	13	4229	546	13	0
	HP LV	4713.7	1261	27	3466.4	784	23	26.5
	Resurf II S	6376.3	1518	23	6700.9	1789	27	-5.1
	Resurf II W	7925.4	2450	31	7532.2	1414	19	4.5

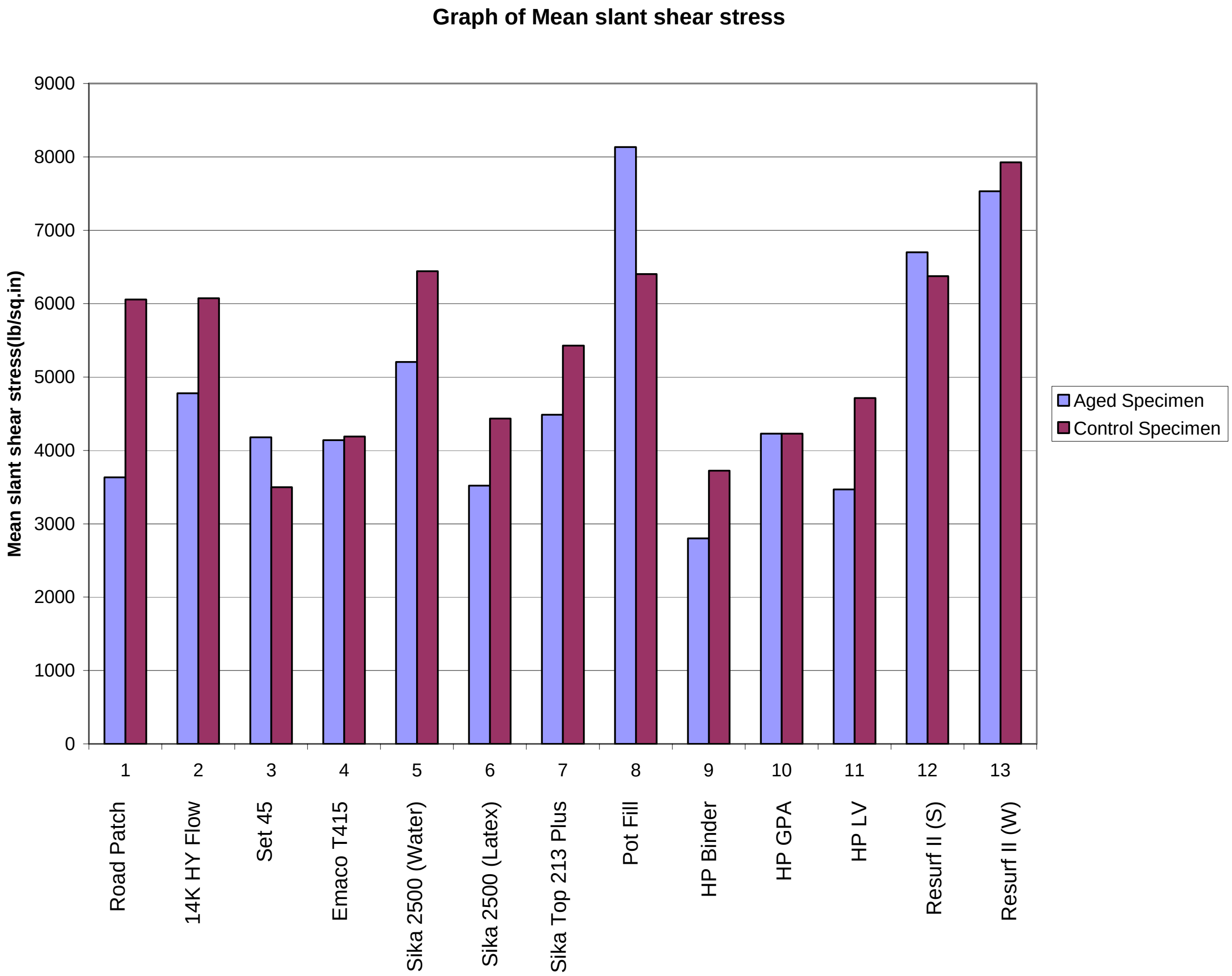


Figure 5.25 *Summary of thermal aging results*

Sample1 MOF(ii)	Sample2 MOF(iii)	Sample3 MOF(i)	Sample4 MOF(iv)	Sample5 MOF(iv)

Figure 5. 26 *Mode of failure of concrete plaques repaired using Road Patch and subjected to 60 thermal cycles*

Sample1 MOF(iii)	Sample2 MOF(i)	Sample3 MOF(ii)	Sample4 MOF(iii)	Sample5 MOF(iv)

Figure 5. 27 *Mode of failure of concrete plaques repaired using 14K HY Flow and subjected to 60 thermal cycles*

Sample1	Sample2	Sample3	Sample4	Sample5
MOF(iii)	MOF(iv)	MOF(iv)	MOF(i)	MOF(i)

Figure 5. 28 Mode of failure of concrete plaques repaired using Set 45 and subjected to 60 thermal cycles

Sample1	Sample2	Sample3	Sample4	Sample5
MOF(iii)	MOF(iii)	MOF(iv)	MOF(iii)	MOF(iii)

Figure 5. 29 *Mode of failure of concrete plaques repaired using Emaco T415 and subjected to 60 thermal cycles*

Sample1 MOF(iv) | Sample2 MOF(ii) | Sample3 MOF(iv) | Sample4 MOF(i) | Sample5 MOF(iv)

Figure 5. 30 *Mode of failure of concrete plaques repaired using Sika Top123 and subjected to 60 thermal cycles*

Sample1 MOF(ii) | Sample2 MOF(iv) | Sample3 MOF(i) | Sample4 MOF(ii)

Figure 5. 31 *Mode of failure of concrete plaques repaired using Sika 2500 (Latex) and subjected to 60 thermal cycles*

Sample1 MOF(iv) | Sample2 MOF(ii) | Sample3 MOF(iii) | Sample4 MOF(iv)

Figure 5. 32 *Mode of failure of concrete plaques repaired using Sika 2500 (water) and subjected to 60 thermal cycles*

Sample1 MOF(iv) | Sample 2 MOF(iv) | Sample 3 MOF(iv)

Figure 5. 33 Mode of failure of concrete plaques repaired using Pot Fill and subjected to 60 thermal cycles

Sample1 MOF(iii)	Sample2 MOF(iv)	Sample3 MOF(iii)	Sample4 MOF(iii)	Sample5 MOF(ii)

Figure 5. 34 *Mode of failure of concrete plaques repaired using HP Binder and subjected to 60 thermal cycles*

Sample1 MOF(iii)	Sample2 MOF(ii)	Sample3 MOF(ii)	Sample4 MOF(ii)	Sample5 MOF(i)

Figure 5. 35 *Mode of failure of concrete plaques repaired using HP GPA and subjected to 60 thermal cycles*

Sample1 MOF(iii) | Sample2 MOF(ii) | Sample3 MOF(iii) | Sample4 MOF(ii) | Sample5 MOF(ii)

Figure 5. 36 *Mode of failure of concrete plaques repaired using HP LV and subjected to 60 thermal cycles*

Sample1 MOF(iv) | Sample2 MOF(iii) | Sample3 MOF(iii) | Sample4 MOF(i) | Sample5 MOF(iv)

Figure 5. 37 *Mode of failure of concrete plaques repaired using Resurf II (S) and subjected to 60 thermal cycles*

Sample1 MOF(iii) | Sample2 MOF(iii) | Sample3 MOF(iv) | Sample4 MOF(iii) | Sample5 MOF(iv)

Figure 5. 38 *Mode of failure of concrete plaques repaired using Resurf II (W) and subjected to 60 thermal cycles*

5.4.2.1 Discussion

In this study, six factors may affect the change in slant shear strength of repaired concrete sections subjected to thermal variation under constant high humidity.

1- Thermal incompatibility between repair materials and concrete. Thus it is expected that cement based materials will have less reduction in slant shear strength.

2- Elastic constant difference between repair materials and the parent concrete. Which is more pronounced for the case resin-based materials as compared to the case of cement based materials. Thus it is expected that cement based materials will have better performance (less stress concentration).

3- Flexibility of used repair material. This will be an advantage of resin based materials which will allow them to accommodate thermal and mechanical stress that may be set at the interface.

4- Though subjecting test specimens to thermal cycling may cause reduction in slant shear strength, due to thermal and mechanical incompatibility between repair materials and concrete, it may also be associated with additional curing (post curing) of repair material which will lead to increase in their strength.

5- Humidity will degrade the interfacial properties as well.

6- Exothermal reactions of these rapidly cured repair materials will set high stresses at the concrete-repair interface

Thus predicting the response of repaired sections after aging is not an easy task. These factors were the reason of not getting conclusive results from this task. Thus additional investigation is needed.

The cement based materials that showed weak bond strength at static room temperature, failed at the diagonal repair material-concrete interface and at a load significantly smaller than the crushing strength for the parent concrete. They showed either no reduction or increase in slant shear strength after being subjected to thermal cycling. This can be attributed to additional hydration of these materials at high temperature. On the other hand, the cement based materials that have strong bond between repair materials and concrete at room temperature (Road Patch, 14K HY Flow and Sika 2500 in water) failed at significantly lower strength than the control specimens with a reduction in Slant Shear Strength that ranged from 19.2 to 40 %.

Materials like Resurf II and Pot Fill which showed high performance and successful repair at static room temperature by failing in the concrete crushing mode leaving the parent concrete and repair material diagonal joint monolithic, show little deterioration of bond strength. This can be attributed to two factors: the relatively low elastic modulus of these materials and continued curing (post curing) of resin based materials at high temperature.

The reduction in the slant shear bond strength of repaired prisms by reinstatement of section and thermal cycles are attributed to the differential strains at the concrete- repair material interface. These differential strains arising from thermal incompatibility between concrete and repair materials, due to significantly different coefficients of thermal expansions, tend to disrupt and weaken the bond strength between repair material and concrete with increasing numbers of thermal cycles. However on the other hand the low elastic modulus of resin based materials and possibility of post curing were the main factors of the unexpected increase in strength of some of the resin based materials after 60 thermal cycles.

5.5 Conclusion

From this limited evaluation of bond integrity between repair materials and concrete, the following materials had the best properties: Pot-Fill, Resurf II, and Sika 2500 mixed with water.

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