NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE

118

DETECTING DEFECTS AND DETERIORATION IN HIGHWAY STRUCTURES

TRANSPORTATION RESEARCH BOARD
National Research Council

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

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PREFACE

A vast storehouse of information exists on nearly every subject of concern to highway administrators and engineers. Much of this information has resulted from both research and the successful application of solutions to the problems faced by practitioners in their daily work. Because previously there has been no systematic means for compiling such useful information and making it available to the entire highway community, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series reports on various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which these reports are useful will be tempered by the user's knowledge and experience in the particular problem area.

FOREWORD

By Staff Transportation Research Board

This synthesis will be of interest to bridge inspectors, maintenance engineers, designers, and others concerned with determining the condition of existing highway bridges. Information is presented on potential defects and deterioration in various components of bridges and on field and laboratory tests that are available to test bridge components.

Administrators, engineers, and researchers are continually faced with highway problems on which much information exists, either in the form of reports or in terms of undocumented experience and practice. Unfortunately, this information often is scattered and unevaluated, and, as a consequence, in seeking solutions, full information on what has been learned about a problem frequently is not assembled. Costly research findings may go unused, valuable experience may be overlooked, and full consideration may not be given to available practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of reporting on common highway problems and synthesizing available information. The synthesis reports from this endeavor constitute an NCHRP publication series in which various forms of relevant information are assembled into single, concise documents pertaining to specific highway problems or sets of closely related problems.

The various types of materials used in constructing highway bridges are subject to defects and deterioration. This report of the Transportation Research Board gives the common defects and types of deterioration encountered in concrete, steel and wrought iron, and timber components of bridges and discusses available field and laboratory procedures that can be used for detecting defects and deterioration.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the researcher in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

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Adrian G. Clary, Engineer of Maintenance, Transportation Research Board, assisted the NCHRP Project 20-5 Staff and the Topic Panel.

Information on current practice was provided by many highway and transportation agencies. Their cooperation and assistance were most helpful.

DETECTING DEFECTS AND DETERIORATION IN HIGHWAY STRUCTURES

SUMMARY

The need to detect defects and deterioration in highway structures ranges from reasons of safety to the prediction of load capacity to the information base for a bridge management system. This synthesis reviews the methods that can be used to detect deterioration in the above-grade components of concrete, steel, wrought-iron, and timber bridges.

The advantages and disadvantages of nondestructive test procedures suitable for field and laboratory use are discussed along with destructive procedures that can be performed in the laboratory on samples removed from structures. Emphasis has been placed, however, on test procedures that can be used in the field and guidance is given in interpreting the significance of measured values. There are only a few scientific principles on which nondestructive test techniques are based. These have been presented to explain the basis, and the limitations, of the many techniques that are available.

Concrete is the most widely used material in highway structures. It is relatively inexpensive and durable. It is also subject to deterioration by cracking, scaling, corrosion of embedded reinforcement, and disruptive chemical reactions between the mixture ingredients or between the concrete and the external environment. Numerous techniques for detecting defects and deterioration have been investigated ranging from striking the concrete surface to detect delamination to the radiography of post-tensioned concrete. Some new applications of technology from other fields, such as infrared thermography and radar, show considerable promise in this area.

Steel is a versatile construction material, much stronger and more homogeneous than concrete and timber. The most common types of deterioration in steel are corrosion and fatigue cracking. There are many investigative techniques available; some, such as radiography and ultrasonics, are better suited to detecting internal defects whereas others, such as magnetic particles, eddy currents, and dye penetrants, are suitable for detecting surface cracks. Significant progress has been made in recent years in refining ultrasonic test procedures. With the development of suitable data-processing techniques, acoustic emission measurements may prove very useful for the long-term monitoring of steel structures.

Timber is strongly anisotropic, having vastly different properties parallel and perpendicular to the grain. It is subject to weathering, decay, and attack by vermin and is vulnerable to fire. Timber highway bridges are relatively few in number and are generally located on secondary highways. Test techniques, of which depth of penetration to test for soundness is the most common, tend to be unsophisticated.

Although it is usually possible to detect deterioration in a component of a structure, it is often difficult to assess the effect of the deterioration on the load-carrying capacity of the structure. In such cases, and where the cost can be justified, full-scale testing

can be performed to measure the overall response of the structure to external stimuli such as static loading or vibration.

Research is needed to evaluate many of the recent developments under controlled conditions, to refine procedures for the full-scale testing of bridges, and to minimize the cost of investigations by matching the capabilities of the test procedures available to the end use of the data. In concrete structures there is a need for techniques that will measure the rate of corrosion of embedded steel; the permeability, resistivity and moisture content of the concrete; and conditions inside ducts containing post-tensioned cables. For steel structures research is needed to develop methods for predicting remaining fatigue life and for inspecting critical components, such as eye-bars, cables, hinge pins, and connections. Research in timber bridges should focus on measuring the in-place strength of timber components.

A survey of current practice revealed that concrete structures are being tested more extensively than steel or timber structures. This reflects both the availability of test methods that are simple yet meaningful and the major deck rehabilitation programs underway in many states. Except for visual examinations, there is little routine testing of steel structures. Most states have few timber structures under their jurisdiction. Testing is rarely done, and when it is, only simple procedures are used.

CHAPTER ONE

INTRODUCTION

The purpose of this synthesis is to describe methods of detecting defects and deterioration in reinforced concrete, prestressed concrete, steel, wrought iron, and timber highway bridges. Its scope is determined by the needs of the practicing engineer involved in the inspection, condition surveys, and evaluation of existing bridges. Emphasis is placed on the underlying principles, the capabilities, the relative advantages and disadvantages, and interpretation of the results of the test procedures, which are described. Although both laboratory and field test methods are included, prominence is given to nondestructive test techniques that are suitable for use in the field. There are a large number of nondestructive test methods available that were developed primarily for quality control testing in new construction. These methods are included only to the extent that they can also be used to assess the serviceability of existing structures.

The scope of the synthesis is also limited to those activities that would normally be performed on the above-grade components of highway bridges. The inspection and testing of components below the waterline is the subject of Synthesis 88 (*1*). Components below grade are the subject of reference *2*. Also excluded are the mechanical and electrical components of moveable bridges. The evaluation of the structural capacity of deteriorated bridges is not discussed in detail but guidance is given in carrying out full-scale load tests. Further information is available from the many references cited. There is considerable information on nondestructive testing techniques available in the literature although much of it is in publications not usually read by engineers having operational responsibility. Furthermore, a number of new techniques have been developed in recent years that hold considerable promise for improving the capabilities to determine the condition of existing structures. The contents of the synthesis have been organized with respect to the various materials used in bridge construction rather than by structural components or type of defect. This has been done because the effectiveness and appropriateness of the test methods tend to be a function of the material rather than its position in the structure.

The extent and detail to which defects and deterioration need to be detected is governed by the end use of the information and by any legal requirements. For example, a visual survey may suffice for the purpose of assessing overall integrity and structural safety. Testing may be required if the existence and approximate amount of deterioration must be known for the purposes of assessing priorities and planning for the future rehabilitation. Yet a third level of investigation must be employed when it is necessary to define the extent and precise location of defects and deterioration for the purposes of determining the types of remedial work necessary and preparing the contract documents.

It is most important that the purpose of the investigation be clearly identified so that the quality of the information is adequate to achieve the desired result. Too much information can be as bad as too little and also a waste of resources. Although the use of instruments can be most advantageous in quantifying defects, it is possible to increase the sensitivity of detection methods to the point where the indication of flaws is either false or confusing. One example of this (*3*) is the use of dye penetrates to examine fine crack patterns in a concrete surface. After considerable effort, it was concluded that the cracks were characteristic of a normal concrete surface and had no stuctural significance. Consequently, the ideal situation is to supplement the trained eye, experience, and the perceptive mind with instruments that are easy to use in the field, are affordable, and yield clear and unambiguous results.

The human resources necessary to test for defects and deterioration are a function of both the test methods used and the physical environment of the structure. Careful planning of the work is always important but particularly so in cases where access is difficult, such as on high-level structures or where traffic volumes are heavy. However, even where special equipment and safety procedures must be used to gain access, it is desirable that remote observations (such as through binoculars) be supplemented by observations at close range. Each agency will normally develop its own requirements for reporting test results but it is advantageous that standard procedures be developed. Further information on the safety, planning, organization, and manpower required for inspection and testing is given in references *4–7* and in the future NCHRP synthesis, Topic 16-01, "Bridge Inspection Practices—Equipment, Staffing, and Safety."

CHAPTER TWO

NONDESTRUCTIVE TESTING TECHNIQUES

INTRODUCTION

This chapter describes the scientific principles upon which testing techniques, and especially nondestructive test methods, are based. This has been done to counter the "black box" approach to testing and permit the reader to gain an appreciation of the capabilities and limitations of the many techniques available. It also provides the background against which to evaluate the probability of success of new techniques and equipment.

The basic principle used in detecting defects not visible to the naked eye is that the defect must cause a change in the response of the interrogating medium. Furthermore, the difference in response between the defect and sound material must be large enough to be recorded on the instrumentation. Most of the nondestructive testing techniques use various waveforms as the interrogating medium. These waves are either electromagnetic radiations or stress waves. The principle characteristic of electromagnetic waves is that they travel freely through a vacuum. Stress waves are transmitted from one molecule to another by direct contact between the molecules; consequently they can only exist in mass media. Stress waves are sometimes termed elastic waves because it is the elastic property of a material that controls the velocity of stress waves through it. Other interrogating media used in nondestructive testing are electricity, magnetism, and charged particles.

ELETROMAGNETIC RADIATION

The electromagnetic spectrum is shown in Figure 1 and comprises wave lengths ranging from approximately 10^{-13} m to several thousands of meters. The common property of the waves comprising the electromagnetic spectrum is that they are not affected by electric or magnetic fields. The velocity of the waves is 3×10^8 m/sec in vacuum. In other media, the velocity is a function of the density and atomic number of the media. The frequency and wavelength are related by the simple expression

$$f = \frac{v}{\lambda} \tag{1}$$

where

f = frequency,
v = velocity, and
λ = wavelength

Although the entire electromagnetic spectrum has a profound effect on everyday life, it is the shorter wavelengths that have application in nondestructive testing because of the greater energy levels associated with them. The ability of radiation to penetrate matter is a function of the kind of matter and the wavelength (or energy) of the radiation. The loss of energy as the radiation penetrates the material is known as attenuation and depends on both the nature of the material (different materials absorb radiation at different rates) and its thickness. Long-wavelength waves, such as broadcast waves, not only require large antenna arrays but attenuate rapidly in solid material.

Microwaves are the longest-wavelength wave in general use for testing and inspection purposes. Microwaves range in wavelength from 300 to 0.3 mm, corresponding to a frequency range of 10^9 to 10^{12} Hz (*8*). Their most common use in materials testing is to determine the moisture content of porous materials. Microwaves are strongly absorbed by water and consequently moisture content can be measured from the attenuation characteristics of the microwaves.

A laser is a pure concentration of coherent light of almost a single wavelength. Most lasers have wavelengths in the visible and infrared ranges but work is underway to produce "free-electron" lasers ranging in wavelength from microwaves to X rays. Lasers are produced when a medium, such as a crystal, gas, or liquid, is energized by high-intensity light, an electric discharge, or nuclear radiation. Energy levels range from very low energy continuous discharges to very high energy pulses. Although lasers have numerous industrial and medical uses, the main application for highway structures is in precision measuring devices and, possibly in the future, holography.

The only difference between X rays and gamma rays is one of origin, although the properties of each are dependent on the particular wavelength and energy (*9*). X rays result from an atomic process outside the atomic nucleus; gamma rays usually from a nuclear process. Together they make up the short wavelength, high-frequency end of the electromagnetic spectrum and have the greatest penetrating power. The frequency is directly proportional to the wave energy so that gamma rays have higher energy levels than X rays. X-ray energies are typically in the kilo electron volt (10^3 eV) range; gamma rays have energies in the mega electron volt (10^6 eV) range. The individual rays are often referred to as photons. Each photon, having a specific energy, can be thought of as having an equivalent mass, the value of which is given by Einstein's equation, $E = mc^2$. The photons behave as, and can be treated as, particles with mass and momentum. It is often more convenient to think of X rays and gamma rays as particles rather than as waves, especially when considering particle attenuation.

A recent development in the field of radiography has been the technique of computed tomography. The process is used to produce an image from thousands of low-voltage X-ray radiographs taken from all sides of an object. It has proven to be a very valuable tool in the medical profession for the examination of tissues not visible on conventional X-ray plates. The same techniques are now beginning to be applied to the study of engineering materials (*1*).

STRESS WAVES

Stress waves can propagate through solid materials and exhibit the four principal properties of wave motion, which are

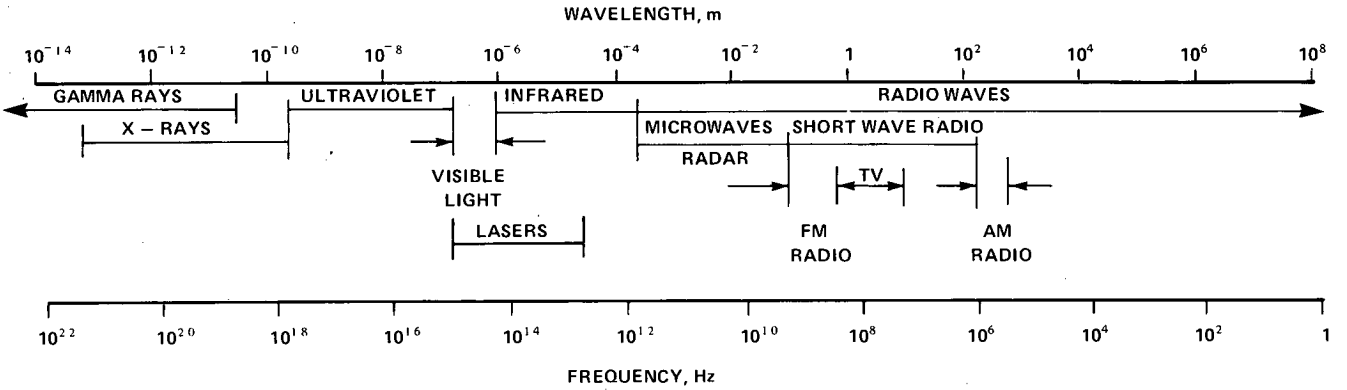

FIGURE 1 The electromagnetic spectrum.

reflection, refraction, defraction, and interference. The ability to use stress waves to detect flaws or discontinuities in a material is predicated on the fact that the boundary between flaws and sound material offers differing resistance to the passage of the waves. The boundary acts as a site for partial reflection (or refraction, defraction, or interference) of wave energy. Thus waves passing through a material strike either a flaw or, eventually, an external surface. The vibrations are reflected and the nature of the return signal indicates the location and type of reflecting surface.

When vibration is propagated through a solid medium, three kinds of waves are generated in the solid. These are classified as longitudinal, transverse, and surface waves. The longitudinal or compression waves are the most important in nondestructive testing and their velocity can be determined from the expression:

$$V_c = \sqrt{\frac{E}{\rho}} \tag{2}$$

where

V_c = velocity of compression wave,
E = dynamic elastic modulus, and
ρ = density.

A more complete mathematical treatment of stress waves is given in Appendix B.

Pulse-velocity measurements are a useful technique for investigating materials because the velocity of the pulses depends only on the elastic properties and is almost independent of geometry. Additional information is provided by virtue of the fact that the elastic properties can be correlated with other mechanical properties.

A wide range of vibration frequencies can be generated ranging from approximately 0.01 Hz to in excess of 10^{12} Hz. The more common methods of generating and receiving vibrations for each frequency range are given in Figure 2. Vibrations at the low end of the frequency range are generated by mechanical disturbances or explosives. The long wavelengths involved mean that waves in the subaudible range can recognize only gross discontinuities and their primary use is in geophysical investigations. At the other end of the scale, waves in the hypersonic range are used in the laboratory to detect minute flaws, such as in the investigation of crystal lattices. Between the two extremes are the sonic and ultrasonic frequency ranges, which are widely used in nondestructive testing in both the field and the laboratory.

Acoustic emission is the term applied to the low-frequency sounds emitted when a material is deformed (*10*). In its simplest form, the emission can be of such a high level that it is audible to the unaided ear. Familiar examples are the creaking of timber when subjected to loads near failure or the striking of a component with an object, such as a hammer, and listening to the response. In the latter case, areas of deterioration are associated with a dull or hollow response. Most materials emit sounds or stress waves as they are deformed and these sounds can provide information on the deformation characteristics and warn of impending failure. However, the sounds are usually of such a low level that sophisticated instrumentation systems are required to detect them.

Sonic testing techniques can conveniently be classified into resonant and ultrasonic pulse measurement methods. The resonance method involves the measurement of the natural frequencies of vibration of a component. The method of causing vibration can be either a single impact excitation or a forced vibration. For a prismatic section, the fundamental frequency of vibration is given by substituting n = 1 in the expression

$$f_n = \frac{n^2}{2L^2}\sqrt{\frac{EI}{m}} \tag{3}$$

where

f_n = frequency of nth mode of vibration,
L = length between supports,
E = dynamic modulus of elasticity,
I = second moment of area of section (moment of inertia),
m = mass per unit length, and
n = mode of vibration.

Eq. 3 is often expressed in the form

$$E = \frac{WK}{T^2} \tag{4}$$

where

W = weight of the specimen,

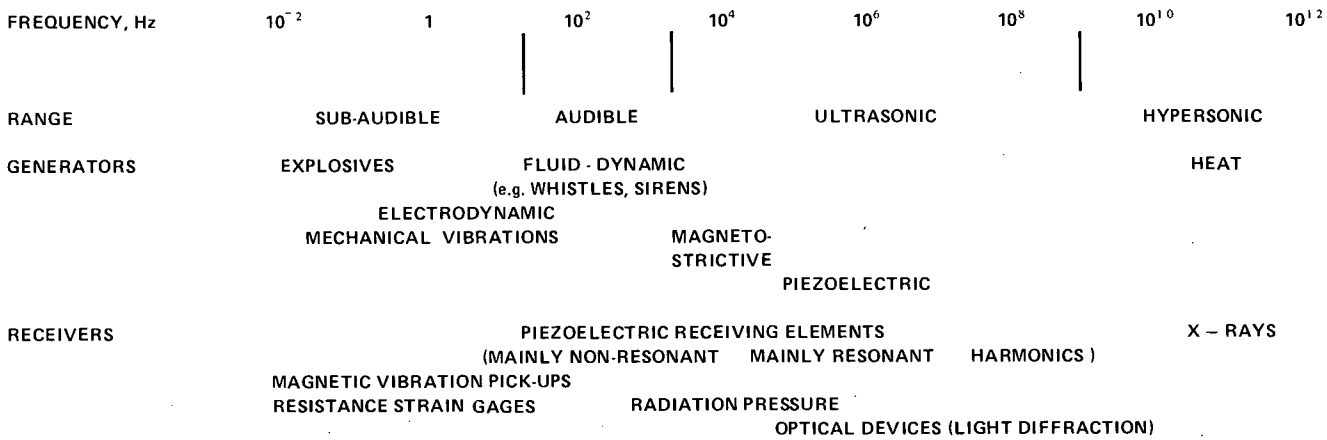

FIGURE 2 The stress wave spectrum.

K = dimensional constant for a particular mode of vibration, and
T = period of transverse vibration.

The use of transverse vibration to detect deterioration depends on a sound specimen having a characteristic mode of vibration whereas the presence of internal defects affects the frequency or amplitude of the vibration. This method of testing is widely used in the laboratory for measuring the freezing and thawing resistance of concrete and in the stress grading of timber. However, the method does not lend itself to field testing. The application of the technique to the full-scale testing of bridges is discussed in Chapter 6.

Ultrasonic testing involves the application of sound waves at frequencies above those of the hearing range of the average person (about 16 kHz). Ultrasonic testing can be divided into two categories, one dealing with low-intensity or low-amplitude vibration and the other with high energies. Low-amplitude applications are those where the primary purpose is transmitting the energy through a medium without changing the state of the medium. A typical application is the measurement of the elastic properties of the material. High-energy applications are those where the purpose is to produce an effect on the medium through which the wave propagates.

OTHER METHODS

The other interrogating media that have application in the nondestructive testing of highway structures are electricity and magnetism and, although of lesser importance, charged particles.

Electrical resistance measurements in concrete and timber are most frequently associated with the measurement of moisture content. In steel components, the excitation of eddy currents and measurements of perturbations in the electrical field is one method of detecting internal defects.

Magnetic methods, because they are applicable only to magnetic materials, are used primarily for the examination of steel structures. In magnetic particle testing, the steel component is magnetized and disturbances in the magnetic field (as revealed by iron powder) indicate the presence of surface or near-surface defects. Other important applications of magnetic testing methods are in the measurement of thickness of nonmagnetic coatings applied to steel and in detecting the location of reinforcement in reinforced or prestressed concrete structures.

Charged particles, such as alpha and beta particles and neutrons, are used in special applications. The particles are provided by the decay of appropriate isotopes. Alpha particles penetrate solid materials poorly and have few applications. Beta particles offer greater penetration, although considerably less than gamma rays, and have limited application in the quality control testing of thin sheet timber products. The principle application of neutron absorption and scattering techniques is in the detection of moisture in concrete and timber. The method is based on the large cross section of the hydrogen nucleus for elastic scattering and, even more important, the nearly equal masses of the neutron and the hydrogen atom (*11*). These conditions allow more energy to be transferred from neutrons to hydrogen than to any other element.

CHAPTER THREE

CONCRETE COMPONENTS

CHARACTERISTICS OF CONCRETE

Concrete is the most widely used material in highway structures. Most bridges include some concrete in foundations, piers, abutments, beams, decks, or walls.

Concrete is made from fine and coarse aggregates bound together by hydrated portland cement. As such, it is the least homogeneous of the common construction materials. The most important property of the cement paste is its porosity because the hydration products of the portland cement never completely fill the space originally occupied by the portland cement and water in the concrete mixture. The size and distribution of the pores within the paste influence many of the more important properties of the concrete including its strength and durability. The most useful attributes of concrete as a construction material are that it is made from relatively low-cost materials that are widely available and it can be cast in any shape. However, the inevitable variations in the quality of the constituents, the mixture proportions, and workmanship also result in the varying performance of concrete components.

Concrete can have a high compressive strength and this is used as a good indicator of its overall quality. Shear and tensile strengths are much lower, being about 12 percent and 10 percent of the compressive strength. Tensile capacity is provided by reinforcing steel so that concrete is suitable for use in flexural members. In prestressed concrete, compression is applied to the concrete by means of highly stressed strands or bars of high-strength steel. This compressive strength is sufficient to offset the tensile stresses caused by applied loads. Prestressing techniques can be divided into pretensioning, where the steel is stressed before the concrete is placed, and post-tensioning, where the steel is stressed after the concrete has hardened. Pretensioned concrete products, of which beams are the most common example in highway structures, are normally manufactured in permanent plants, whereas post-tensioning is normally a field operation.

Concrete itself is generally isotropic but the addition of reinforcement makes it anisotropic (i.e., the strength varies with the direction of loading). Concrete is a poor conductor of electricity and heat. It is much more resistant to fire than steel or timber but intense heat will damage concrete and cause spalling of the cover from the reinforcement. Although concrete is an elastic material under ordinary load, the strain at the proportional limit is small. Sustained load results in creep. Its low tensile strength means it is susceptible to cracking and the width of the cracks must be controlled by reinforcement. An important characteristic of concrete is the high internal humidity and the constant exchange of moisture between the concrete and its environment, which results in relatively large volume changes. Where the concrete is restrained, the volume changes resulting from changes in humidity or temperature can result in cracking of the component.

The durability of concrete is determined by its quality and the exposure conditions. In general, quality is primarily a function of the water-cement ratio. As the water-cement ratio (and consequently the porosity) decreases, durability increases provided that the concrete has been made from sound materials and has been properly proportioned, consolidated, and cured. Air entrainment is required for resistance to freezing and thawing under wet conditions. The biggest problem in the durability of concrete in highway structures in recent years has been the corrosion of embedded reinforcing steel, which results from chloride ions penetrating the concrete (*12*).

COMMON DEFECTS AND TYPES OF DETERIORATION

The most common forms of defects and deterioration that occur in concrete components are described below. Photographs of the defects are contained in references *4*, *6*, and *13*.

Cracking

A crack is defined as an incomplete separation into one or more parts, with or without a space between them. The significance of cracks in concrete is dependent on their origin and whether the length and width increase with time. There are several possible causes of cracking in concrete structures. The most common causes and characteristics of the cracks resulting from each cause can be summarized as follows (*5*):

- Plastic shrinkage cracks result from rapid drying of the concrete in its plastic state. The cracks are usually wide but shallow and often in a well-defined pattern or spaced at regular intervals.
- Drying shrinkage cracks result from the drying of restrained concrete after it has hardened. They are usually finer and deeper than plastic shrinkage cracks and have a random orientation.
- Settlement cracks may be of any orientation and width, ranging from fine cracks above the reinforcement that result from settlement of the formwork to wide cracks in supporting members caused by foundation settlement.
- Structural cracks, except for those controlled by the provision of reinforcement, result from differences between assumed and actual stress intensity. The width varies but the orientation is often well defined. Examples are diagonal cracks in the acute corners of severely skewed decks and longitudinal cracks over internal voids in some thick slab decks.
- Map cracking (a closely spaced network of cracks) usually results from chemical reactions between the mineral aggregates and the cement paste. The number and the width of the cracks usually increases with time. A number of reactions are possible although the reactions between the alkalis from the cement, or from external sources, and two constituents of some aggregates

(producing alkali-silica or alkali-carbonate reactions) are the most widespread. Both types of reaction result in serious damage to the concrete by causing abnormal expansion, cracking, and loss of strength (*14*).

- Corrosion-induced cracks (resulting from the corrosion of embedded reinforcement) are usually associated with shallow cover and are located directly above the reinforcement. The cracks terminate at the reinforcement and rust stains may be associated with them. The width of the cracks increases with time as the corrosion continues.

Scaling

Scaling is the flaking away of the surface mortar of the concrete. As the scaling progresses, coarse aggregate is exposed and eventually loosened. Scaling results from the repeated freezing of concrete that is critically saturated with water and is aggravated by the presence of deicing salts.

Scaling may occur in a weak surface layer resulting from poor finishing or curing practices (including construction in the rain), in which case it may progress no deeper. More commonly, it is indicative of inadequate air entrainment and may ultimately progress to complete disintegration of the concrete. Scaling is more common in older structures and especially in asphalt-covered decks that are not waterproofed because the bituminous surfacing prevents the concrete deck slab from drying.

Corrosion of Embedded Reinforcement

Corrosion of reinforcement is the electrochemical degradation of steel in concrete. It occurs when oxygen and moisture are present and the passivity of the steel is destroyed by either carbonation of the concrete or by the presence of more than the threshold concentration of chloride ions at the steel surface.

Carbonation of concrete is the result of the reaction of carbon dioxide and other acidic gases in the air (which form weak acids in solution) and the alkaline constituents of cement paste. As a result, the alkalinity of the concrete is reduced and the steel is no longer protected against corrosion. The depth of carbonation increases with age but is a very slow reaction, typically no more than about 0.04 in. (1 mm) a year. The rate is highest in structures with a high water-cement ratio and in a dry environment. Carbonation has not been identified as a serious problem in highway structures and corrosion is normally initiated by the ingress of chloride ions and not carbonation.

As the steel corrodes, it expands and causes a delamination (a separtaion along a plane parallel to the surface of the concrete) usually located at, or near, the level of the reinforcement. As the corrosion processes continue, delaminations eventually become detached from the concrete body resulting in a spall. Cracks are not necessary for the occurrence of corrosion because chloride ions will penetrate uncracked, high-quality concrete. Depending on the amount of cover and degree of corrosion, cracks may, or may not, be present within or at the extremities of delaminated areas.

Honeycombing and Air Pockets

These defects may be present in formed surfaces and originate at the time of construction. Air pockets result from inadequate consolidation. Honeycombing occurs when the mortar does not fill the spaces between the coarse aggregate particles. It may be caused by either incomplete consolidation or leakage of the mortar between sections of the formwork.

Popouts

Popouts are shallow depressions that result from the breaking away of the concrete surface because of internal pressure. They are most frequently associated with frost-susceptible aggregates. Generally, a shattered aggregate particle will be found at the bottom of the hole with a part of the particle still adhering to the popout cone.

Surface Deposits

The most common types of surface deposit are efflorescence and exudation. Efflorescence is a deposit of salts, usually white, which results from the flow of a solution (mainly dissolved calcium hydroxide) from within the concrete to the surface where the water evaporates. Exudation is the solid or gel-like component of material discharged at an opening in the concrete surface. Both efflorescence and exudation may be associated with cracks although this is not always the case.

Chemical Attack

The most widespread form of chemical attack is by sulfates. Although more localized, attack by acids and ammonium salts can be serious where it occurs. Naturally occurring sulfates of sodium, potassium, calcium, or magnesium are sometimes found in soil or dissolved in groundwater adjacent to concrete structures and they can attack concrete. When evaporation can take place from an exposed face, such as an abutment or wing wall, the dissolved sulfates may accumulate at that face and increase the potential for deterioration. The sulfates combine with free calcium hydroxide in the cement to form calcium sulfate, which, in turn, combines with hydrated calcium aluminate to form calcium sulfoaluminate. Both these reactions result in an increase in solid volume, which disrupts the concrete and ultimately causes it to disintegrate.

The deterioration of concrete by acids is primarily the result of a reaction between these chemicals and calcium hydroxide. In most cases the reaction results in the formation of water-soluble calcium compounds, which are then leached away. Although acid attack of concrete in highway structures is not common, it can occur as a result of bird or animal droppings. In such cases deterioration is not simply the effect of organic acids on the concrete alone, but, because droppings and nesting material trap moisture (often containing chlorides), acid attack is found in combination with scaling and even corrosion of embedded reinforcement.

Ammonium salts, often found in farm fertilizer, are particularly aggressive to concrete, even in low concentrations (*14, 15*).

Wear

Excessive wear may be present on concrete surfaces exposed to traffic abrasion and is usually concentrated in the wheel

tracks. Serious rutting may be present where studded tires are permitted (*16*). Wear can sometimes be difficult to distinguish from grinding (used at the time of construction to achieve an acceptable surface tolerance) and light-scaling. Curb areas may be subject to the scraping action of snowplows.

Erosion

Ice movement or, in the case of rivers with high bedload, abrasion by solid particles causes erosion of the surface of piers or abutments exposed to river or lakes. Similar deterioration occurs in marine structures where erosion may be combined with cavitation damage from wave action.

FIELD PROCEDURES

Numerous nondestructive test procedures are available for use on concrete components. Some of these methods are used to indicate strength, whereas others are used to define the nature and the extent of defects and deterioration. Where the equipment and procedures have been standardized, the applicable test methods are listed in Table 1.

TABLE 1

STANDARD ASTM AND AASHTO TEST METHODS FOR USE IN THE FIELD.

Designation[a]	Title
C 42 T 24	Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C 597	Test Method for Pulse Velocity Through Concrete
C 803	Test Method for Penetration Resistance of Hardened Concrete
C 805	Test Method for Rebound Number of Hardened Concrete
C 823	Practice for Examination and Sampling of Hardened Concrete in Constructions
C 876	Test Method for Half Cell Potentials of Reinforcing Steel in Concrete
D 3633	Test Method for Electrical Resistivity of Membrane-Pavement Systems

[a] ASTM test methods are designated C or D.
AASHTO test methods are designated T.

Visual Inspection

The first step in making a condition survey of a structure is to determine the type and the extent of deterioration by a careful visual inspection. The results of the examination are normally reported on standard forms and photographs are taken of significant items. In most cases, access to the surface of the concrete is possible, with the most significant exception being asphalt-covered bridge decks. In this case, the condition of the bituminous concrete may or may not give a reliable indication of the concrete deck slab and experience is needed to recognize the clues that may be present. Bituminous concrete overlays are susceptible to cracking, ravelling, rippling, and rutting such that they require replacement after approximately 15 years, often less under conditions of heavy traffic loadings. Photographs of defects in bituminous concrete and a description of the mechanisms involved are given in reference *17*. It is possible to observe extensive deterioration in the bituminous concrete while the deck slab is in good condition, especially where an effective waterproofing membrane has been provided. Conversely, where the bridge deck has not been waterproofed effectively, the bituminous wearing course may be in good condition and the concrete badly deteriorated. Sometimes concrete stains, radial cracks, or local depressions in the bituminous surfacing are clues to the presence of deterioration in the deck slab. Except where permanent steel forms are present, visual examination of the underside of the deck slab also gives a good indication of its overall condition (*12*). However, it is much more difficult to determine the general condition of asphalt-covered decks than exposed concrete decks by visual inspection; selective removal of the surfacing usually will be necessary.

The most common forms of deterioration that can be identified in a visual inspection are described above. In addition, careful observations should be made for signs of collision damage, excessive deformation, deflection, or vibration.

In carrying out a visual inspection, it is usual to describe cracks with respect to their location, orientation, and width. Depth is also important but can only be determined by coring except where the crack is full depth and therefore visible on opposite surfaces of the member. If the continuity of cracks between surfaces is uncertain, it can be checked using dyes (*18*). The following terms are used to describe the orientation of a crack with respect to the major axis of a member: longitudinal, transverse, diagonal, and random. On vertical surfaces, the terms vertical and horizontal are often substituted for longitudinal and transverse.

Precise measurement of crack width is not usually warranted. If necessary, crack widths can be measured to an accuracy of 0.001 in. (0.25 mm) using a crack comparator, which is a small, hand-held microscope with a scale marked directly on one lens (*19*). A suitable scale for describing crack widths is (*12*):

hairline: less than 0.004 in. (less than 0.1 mm)
narrow: 0.004 to 0.01 in. (0.1 to 0.3 mm)
medium: 0.01 to 0.03 in. (0.3 to 0.7 mm)
wide: greater than 0.03 in. (greater than 0.7 mm)

Care is required to ensure that crack widths are estimated with reasonable accuracy. Ravelled edges or moisture associated with the crack may make the crack more visible and appear wider than it is. Conversely, narrow cracks are difficult to identify on surfaces that are rough or uniformly wet or dry.

Where necessary, crack movement can be measured in a number of ways (*6, 19*). When it is known that movement is perpendicular to the crack, several types of transducer (most notably linear variable differential transformers or LVDTs) or extensometer can be used. Alternatively, crack movement indicators, which give a direct reading of crack translation and rotation, can be attached to the surface of the concrete. These devices are relatively inexpensive and suitable for long-term measurements.

Scaling should be reported with respect to its location and

severity. A suitable scale for classifying scaling is in terms of its depth and characteristic appearance (*4, 5, 12*).

light: 0 to ¼ in. (0 to 5 mm) coarse aggregate not exposed
medium: ¼ to ½ in. (5 to 10 mm) coarse aggregate exposed
heavy: ½ to 1 in. (10 to 25 mm) coarse aggregate projecting from the surface
severe: over 1 in. (over 25 mm) loss of coarse aggregate particles

Corrosion of reinforcement can sometimes be detected by rust stains on the concrete surface. However, care must be taken to avoid confusion from ferrous sulfide inclusions in the aggregate or the rusting of tie wires (*20*). Delaminations are not usually visible except where they are very shallow and there is a discoloration associated with them as is sometimes the case on bridge decks. Spalls are, of course, easily recognized and the reinforcing steel is often exposed within the spalled area.

Inspection of Closed Cells and Inaccessible Areas

It is sometimes necessary to supplement visual inspection by drilling holes to inspect areas inaccessible to the naked eye. Although this situation can occur in any type of structure, it is most common in concrete structures and especially in pretensioned box beams and voided post-tensioned thick slab decks, which are usually built without inspection hatches. A small hole (no more than 1 in. or 25 mm in diameter) permits examination of the interior of closed voids using a tubular instrument known as a borescope. Light is transmitted through a bundle of optical fibers to illuminate the areas of interest and the image is transmitted back to the eye through a lens system. Camera attachments enable photographs to be taken. The same instrument has also been used to examine post-tensioned cable ducts for the presence of voids in the grout (*21*). The major disadvantage of the borescope is that the angle and the field of view are shallow and hence visibility is limited.

Larger holes, made with a core drill, enable visual observations to be made with the assistance of an electric light source and a mirror or periscope. This technique has been used to examine inaccessible areas between the ends of beams and the ballast wall of the abutment (*22*). Video cameras can also be used although it is difficult to find equipment ideally suited to the task. Cameras used in sewer inspections are too large for this work and smaller units tend not to be sufficiently rugged for field use. The defects and deterioration to be recorded are essentially the same as in any other visual inspection (particularly cracks, scaling, spalling, and areas of honeycomb).

Rebound and Penetration Methods

Rebound and penetration methods are used to measure hardness. Although hardness is not itself an important property of concrete, a number of methods of measuring hardness have been developed as a means of predicting the strength of concrete.

Most surface hardness test methods involve indenting the surface of the concrete in a standard manner and measuring the size of the indentation in the same way as the hardness of metals is measured. Four devices have been described (*11, 23*) all of which are of European origin and use a ball as the indentor. Although the test methods are easy and quick to perform, they are not very accurate. Their main use has been in Germany (*24*) and they have not been accepted in North America.

An alternative method of measuring surface hardness is the rebound method. The only known instrument using this principle is the Schmidt Rebound Hammer (sometimes called the Swiss hammer), which was developed in 1948 and has been widely used around the world. The instrument measures the rebound of a spring-loaded plunger as a percentage (called the rebound number) of the initial extension of the spring. The instrument is inexpensive, rugged, and easy to use but has serious limitations. There is little theoretical relationship between the strength of concrete and the rebound number but empirical correlations have been established. The rebound number is affected by many factors including the angle of test, surface smoothness, mix proportions, type of coarse aggregate, the moisture content of the concrete, and carbonation of the surface (*25*). The rebound hammer should be calibrated for each possible position, vertically down, vertically up, or horizontally. Several readings should be taken in each area of interest, being careful to avoid readings directly on coarse aggregate particles (*26*). The rebound hammer is not recommended for use in concrete having a compressive strength of less than 1000 psi (7 MPa). The accuracy of a properly calibrated hammer in predicting the strength of concrete in an existing structure is probably no better than ± 25 percent (*23*). It has been claimed (*27*) that the accuracy can be improved substantially if the rebound hammer is used in conjunction with ultrasonic testing. This may be feasible in quality control testing but is rarely practical on existing highway structures. Consequently, the instrument is most useful in checking the uniformity of concrete in a structure with a view to identifying areas requiring further investigation. It is not suitable for predicting concrete strength in absolute terms unless it is calibrated by taking cores at a number of locations where rebound measurements have been made (*28*).

An extension of the rebound method is to use a more powerful impact device that penetrates the concrete. Such a device, of which the Windsor Probe is the best known example, then measures the hardness of the concrete and not simply the surface hardness. The development of the Windsor Probe began in about 1964 and the equipment consists of a drive unit, or gun, and hardened alloy probes, ¼ in. (6 mm) in diameter. The probes are driven into the concrete by firing a precision powder charge that develops an energy of 575 ft-lb (780 J). The test procedure for field use is to fire three probes into the concrete, one at each corner of a 7-in. (175-mm) equilateral triangle. The exposed lengths of the individual probes are measured by a calibrated depth gauge and averaged. Alternatively, a mechanical averaging device supplied by the manufacturer can be used. Although the manufacturer provides tables relating exposed length of probe to the compressive strength of concrete for different values of aggregate hardness, the reliability of these relationships has been questioned (*23, 29*). A major drawback to the method is that while the coarse aggregate has a major effect on the penetration, it may have little relationship to the strength of the concrete (*26*). As a result, calibration with the strength of the concrete in the structure being tested is necessary.

The Windsor probe penetrates up to 2 in. (50 mm) into the concrete so that the results are less influenced by surface mois-

ture, texture, and carbonation than are the results of the Schmidt hammer. However, the size and distribution of the coarse aggregate in the concrete affect the probe results to a much greater degree than the rebound hammer. The Windsor probe also has the disadvantages that it is more expensive than the rebound hammer and there is a recurring expense for the probes. Also, it is not truly a nondestructive test because it often shatters the concrete surface, and the probes must be removed and the surface patched. Nevertheless, the instrument has been found useful in examining concrete components, such as bridge piers, where coring would have been very difficult (*30*).

The accuracy of the Windsor Probe and Schmidt hammer in predicting concrete strength is comparable. Consequently, the rebound hammer is better suited for use on highway structures by virtue of the fact that it is cheaper, quicker to use, and less destructive. However, both pieces of equipment must be calibrated at each structure investigated if they are to be used for other than identifying anomalous areas.

Stress Wave Methods

The fundamental principles of the transmission of stress waves through materials are given in Chapter 2. These relationships were derived assuming the medium to be homogeneous, isotropic, and perfectly elastic. They may also be applied to heterogeneous materials, such as concrete, provided that the dimensions of the specimen are large compared to the constituents of the material. As a general rule, the least dimension of the specimen should be at least three times the minimum aggregate size and not less than the wavelength of the stress wave (*31*).

The basis of the dynamic test methods for concrete is that the wave velocity is related to the elastic modulus, which is a function of the void content and hence is related to concrete strength (*11*). Thus there is some basis for empirical relationships between wave velocity and strength.

The velocity of stress waves in a solid can be measured by either determining the resonant frequency of specimens or by recording the time of travel of short pulses or vibrations passing through the specimen. Resonant frequency methods are used in the laboratory, especially in testing the freeze-thaw resistance of concrete, and have little application to structures in the field, except in the context discussed in Chapter 6.

Pulse-velocity methods can be conveniently divided into two categories:

1. Mechanical sonic pulse-velocity methods. These involve measurement of the time of travel of longitudinal waves generated by a single impact hammer blow or by repetitive blows.
2. Ultrasonic pulse-velocity methods. These methods involve measurement of the time of travel of electronically generated mechanical pulses through concrete, the time interval being measured by a digital meter and/or a cathode-ray oscilloscope.

Sonic Pulse-Velocity Methods

The mechanical sonic pulse-velocity methods were first applied to the study of concrete pavements (*32*) in the 1940s. The basic principle used was that a longitudinal wave was initiated by a single hammer blow and the time of travel between pickups placed on the pavement surface was measured. The interval time was found to correlate well with the flexural strength of the concrete, which is the property of greatest interest to pavement engineers. The disadvantages of the method were that the measurement of interval times was rather crude and tedious and that the results gave information on the surface condition of the concrete rather than the pavement slab as a whole. Other investigators made improvements to the method by generating pulses at regular intervals, using more sensitive receivers, and displaying the results on a cathode-ray oscilloscope (*33*). Nevertheless, it was found (*34*) that even within the audible range, low-frequency, high-amplitude vibrations, such as those produced by a hammer, had a greater tendency to cross cracks than high-frequency, low-amplitude vibrations.

Mechanical pulse-velocity methods also include microseismic techniques, which have been applied to the detection of deterioration in concrete, especially in asphalt-covered deck slabs. Microseismic refraction involves miniaturizing geophysical techniques so that the survey lines are only a few feet (or meters) long while making a corresponding increase in the sensitivity of the receiving transducers so that travel time can be recorded in micro seconds. Refraction methods are used to compare the travel time between waves travelling near the surface and those refracted from an underlying material that is capable of transmitting the longitudinal wave at a velocity higher than the surface material. Underlying materials with a wave velocity lower than the surface material cannot be detected. The technique was investigated in the 1960s for measuring asphalt thickness and detecting deterioration in a concrete deck slab (*35*). A small, hardened steel sphere was used as a coupler and when struck by a hammer, acted as the triggering device for the timing instrument. A field trial produced encouraging results; self-contained equipment was developed but the method has not been used extensively.

A later investigation (*36*), undertaken to compare the ability of different methods to identify deterioration in asphalt-covered deck slabs, also included an examination of microseismic methods. Although the procedure was capable of detecting differences between known areas of sound and unsound concrete, it was considered impractical because it was time-consuming, interpretation of the data was difficult, and it was not possible to define the extent of the deterioration. Further development was not undertaken because, even with improvements, it was judged unlikely to outperform methods such as radar and thermography, which offered the dual advantages of speed and being noncontact.

A more familiar application of stress waves in the audible range is the acoustic impact method of detecting delaminations by striking the concrete surface and listening to the response. Many tools and instruments have been devised for detecting delaminations. The first were hammers and iron rods, then chains, and, more recently, electronic equipment.

The use of a hammer is tedious because only a small area of the member is investigated at a time. On horizontal surfaces, such as decks, there is considerable operator discomfort but on vertical surfaces there is currently no alternative that is practical and reliable. However, operator fatigue affects the accuracy of the survey as the constant sounding tends to reduce the operator's sensitivity to changes in tone. An iron bar allows the operator to stand upright on decks but otherwise has no ad-

vantages over a hammer. A chain is much more satisfactory because it is simple and economical to use on exposed horizontal surfaces, which can be traversed quickly and accurately (*37, 38*). Chain-drag surveys on asphalt-covered decks are not very accurate but, because they are quick and inexpensive, are useful in identifying anomalous areas, which can then be investigated more thoroughly by other methods (*5*).

A number of configurations have been used for the chain-drag equipment. One form of apparatus is constructed from four or five segments of 1-in. (25-mm) chain about 18-in. (0.5-m) long attached to a 2-ft (0.6-m) piece of copper or aluminum tubing by means of a nonmetallic flexible connection. A handle is attached to the midpoint of the tube to form a "T." Some authorities have found that heavier chains, typically with 2-in. (50-mm) links made from ⅜-in. (10-mm) diameter steel, produce more accurate results, especially when the survey must be carried out with interference from traffic noise (*38*). The usual procedure is to use a chain that is about 6-ft (1.8-m) long until areas of delamination are encountered and then use a much shorter length of chain in contact with the deck surface to define the limits of the delaminated area (*12*).

Although the chain drag is a convenient method of locating delaminations, recording the location and size of the delaminations is very time-consuming. To overcome this disadvantage and eliminate subjective judgment on the part of the operator, a portable electronic instrument was developed specifically for bridge-deck surveys (*39, 40*). The equipment, known as a Delamtect, consists of three basic components: a tapping device, a sonic receiver, and a system of signal interpretation. As the instrument is wheeled across the deck, acoustic signals are generated, propagated through the concrete, received, and interpreted electronically. The output is presented as a waveform on a two-channel chart recorder. Software has been developed that will generate a plan of the deck showing delaminated areas directly from the machine output. The instrument has the advantages that it eliminates the need to mark out a full grid on the deck, it is not affected by traffic noise, and the workload is independent of the number of delaminations present. The main disadvantage is that it is not as accurate as the chain drag and it has therefore been suggested that it is more useful in conducting general surveys when an overall indication of deck condition is required, rather than for isolating specific areas requiring repair (*12*). Attempts have been made to adapt the equipment so that it can be used on vertical surfaces but none are known to have been successful. It has also been claimed that the instrument will detect delamination up to 4.5 in. (115 mm) below the surface of decks with an asphalt wearing course (*40*). Other work has shown the instrument to be unsatisfactory on asphalt-covered decks, partly because of difficulties in calibration and partly because the equipment cannot distinguish between delaminations in the deck slab and poor bond between the deck slab and the bituminous overlay (*36*).

The use of more sophisticated low-level acoustic emission techniques has been investigated in recent years. Acoustic emissions are minute perturbations in stress waves created by localized deformations in the concrete and detected by piezoelectric sensors attached to the concrete surface. The variations in the time of arrival of the stress waves at each sensor are used to locate the source of the deformation (*41*). Although the method has potential for such applications as in-situ strength prediction and detecting crack growth (*42*), there are still serious technical difficulties to be overcome. Acoustic emission has been used, on an experimental basis, in England to determine the in-situ strength of high-alumina-cement beams in buildings (*43*). It has also been used experimentally to detect stress corrosion on strands in a prestressed concrete beam (*44*). Laboratory tests were encouraging but field testing proved inconclusive.

Analysis of acoustic emission data from a structure as complex as a bridge is very difficult and there are practical problems in applying load to stimulate emissions without damaging the structure (*45*). There are also difficulties in eliminating extraneous noise and in recognizing defects. However, most of these problems can be expected to be overcome as more experience is gained and as equipment and data-processing methods are refined.

Ultrasonic Pulse-Velocity Methods

The development of equipment for making ultrasonic pulse-velocity tests began in the 1940s (*46, 47*). The equipment, which was known as a soniscope in North America, was the subject of numerous investigations during the subsequent years (*48–52*).

Further development of the equipment in the 1960s succeeded in producing ultrasonic testers that were small, battery operated, and fully portable, and that had a digital readout. They were much better suited to field use than the heavy soniscopes, which were more suited to research studies and required trained personnel to operate (*49*). Of the commecial units available the PUNDIT (portable ultrasonic nondestructive digital indicating tester) from Britain and the V-Meters, made in the United States, are probably the best known in North America.

The method consists of measuring the time of travel of an ultrasonic pulse passing through the concrete being tested. The pulses are generated using electronic circuitry and transformed to mechanical energy by a transducer containing piezoelectric crystals. This results in vibration frequencies that are generally in the range of 20 to 50 Hz, depending on the specific model of equipment, although frequencies as high as 150 Hz have been used, especially in laboratory studies. The pulses are repeated at the rate of 50 to 150 per second. Although the high frequencies are more sensitive to detecting voids and can be used with much thinner specimens, they are also subject to greater attenuation. Equipment operating at a frequency of 50 Hz is not recommended for use on sections less than 6-in. (150-mm) thick and 20-Hz equipment should be limited to use on sections more than 12-in (300-mm) thick (*31*).

Contact with the concrete is made with a suitable acoustic coupling medium, such as a medium grease, petroleum jelly, liquid soap, or a kaolin-glycerol paste. The coupling medium should be as thin as possible. If the concrete surface is very rough, it may be ground or a smooth surface may be made by using a thin layer of plaster of paris, a quick-setting epoxy mortar, or other suitable material. A similar transducer is coupled to the concrete at a measured distance from the transmitting transducer to act as the receiver so that the time of travel between the two is measured electronically.

There are three ways of measuring pulse velocity through concrete and these are illustrated in Figure 3. In the direct transmission method, the transducers are attached to opposite faces of the member. This method is preferred wherever access

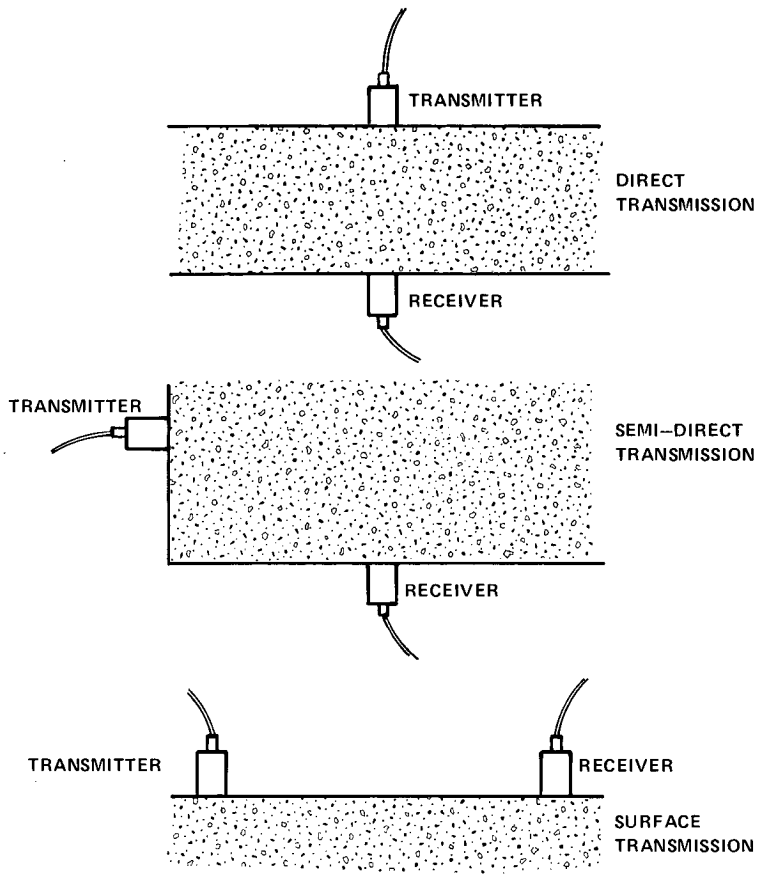

FIGURE 3 Methods of measuring pulse velocity through concrete.

to opposite sides of the component is possible because it provides a well-defined path length and results in maximum sensitivity. Sometimes the geometry of the component (for example, an abutment) requires that the semi-direct method illustrated in Figure 4 be used. Surface transmission is the least satisfactory of the three methods and should be used when access to only one surface is possible. Not only is the maximum energy of the pulse being directed into the concrete but the method only indicates the quality of the concrete near the surface and is influenced by the presence of reinforcement parallel to the surface.

Pulse-velocity measurements have been found to be a reliable indication of the overall quality of concrete. This is because the wave velocity is reduced by the presence of porosity (which affects strength) and internal cracking (which is often associated with deterioration). A number of qualitative scales have been published to relate pulse-velocity measurements to the quality of concrete. Examples are given in Table 2. Although there is agreement that high velocities are associated with concrete of good quality and low velocities with that of poor quality, there is no sharp delineation between the various categories. The data in Table 2 are, however, useful as guides when interpreting results.

Measurements of pulse velocity through concrete are affected by the smoothness of the concrete surface, concrete temperature, moisture content, mix proportion, age of the concrete, and presence of reinforcing steel. Temperatures within the range of 40 to 85° F (5 to 30° C) do not affect pulse-velocity measurements significantly. Outside this range, corrections can be applied (*53*). An increase in moisture content increases the pulse velocity but generally by not more than 2 percent.

The presence of reinforcing steel affects the pulse-velocity measurements considerably because pulse velocity in steel is 1.2 to 1.9 times the velocity in plain concrete. Generally, it is desirable to choose path lengths that avoid the influence of the reinforcing steel. Where it is not possible to do so, the measured values have to be corrected. Where the axis of the reinforcing bars is perpendicular to the direction of wave propagation and the quantity of reinforcement is small, the influence of the reinforcement is also small. Where the axis of the reinforcing bars is parallel to the propagation of the pulse, the influence of the reinforcing bars can be substantial. Methods of calculating correction factors are usually contained in equipment manuals and simple procedures are given in references *23, 31,* and *53*. Some theoretical considerations and more complete approaches are discussed in references *54* and *55*. In the case of two-way reinforcement parallel to the pulse, it is almost impossible to make reliable corrections. Furthermore, the correction factors assume a knowledge of the size and location of the reinforcement, which is not always the case for structures in the field.

There appears to be reasonably good correlations between pulse velocity and compressive strength, which enable the concrete strength to be predicted within ± 20 percent provided a calibration curve is established for the concrete being investigated. It is not possible to predict the strength of concrete of unknown composition. Pulse-velocity measurements can be used

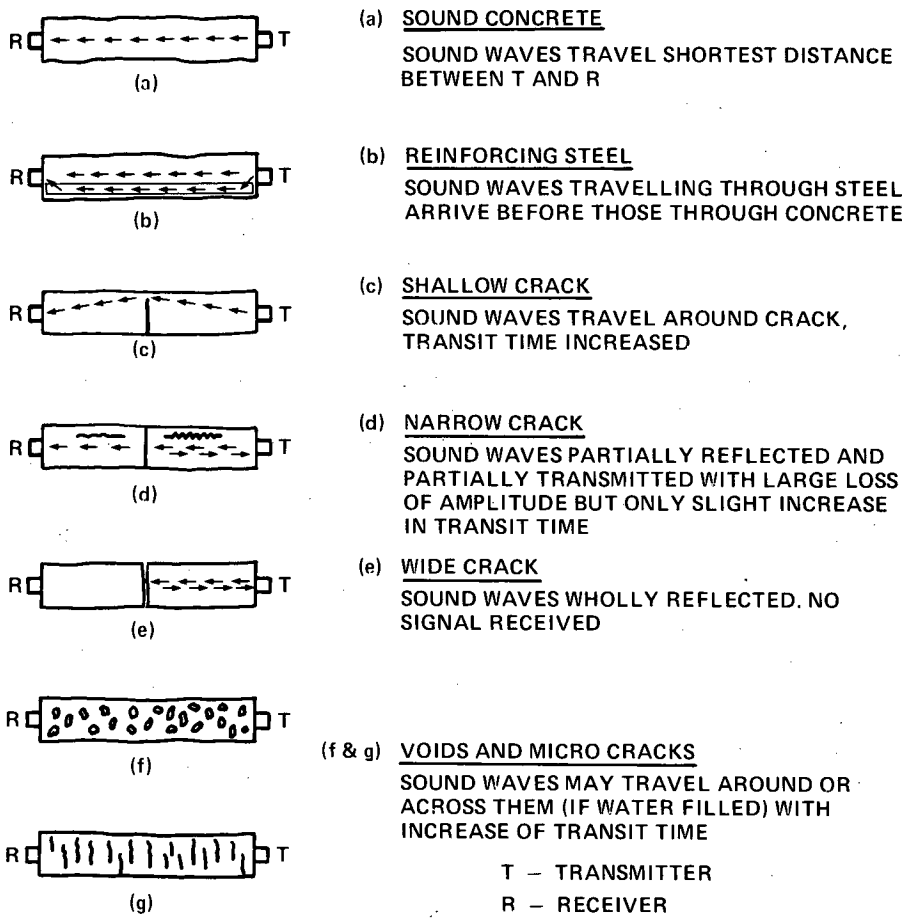

FIGURE 4 Factors affecting the transmission of sound wave through concrete.

to detect voids (*56*) and cracks providing that the crack is approximately perpendicular to the propagation of the pulse. It has been suggested that the transmission of ultrasonic waves is negligible across an air-filled crack more than 0.001-in. (0.025-mm) wide and transmission across a water-filled crack is only about 4 percent of that through uncracked concrete (*46*). A more recent study found that air-filled surface cracks 0.002-in. (0.05-mm) wide and 1.5-in (40-mm) deep could be detected in plain concrete but similar cracks 3/4-in. (19-mm) deep could not be clearly detected under laboratory conditions. Crack detection capability under field conditions is expected to be much less (*57*). Large internal cracks in mass concrete were successfully detected using the soniscope by either an abnormally long transmission time or by a large decrease in the amplitude (*58*). Amplitude assessment techniques can, of course, only be used when the test equipment has an analog display. A summary of the conditions that influence the transmission of sound waves in concrete is given in Figure 4 (*59*).

TABLE 2

PULSE-VELOCITY RATINGS FOR CONCRETE

	Pulse Velocity			
	After Malhotra (23)		After Leslie and Cheesman (46)	
Quality	ft/sec	km/s	ft/sec	m/s
Excellent	>15,000	>4.6		
Good	12,000 to 15,000	3.7 to 4.6	> 16,000	> 5.0
Fair	10,000 to 12,000	3.0 to 3.7	13,000 to 16,000	4.0 to 5.0
Poor	7,000 to 10,000	2.1 to 3.0	10,000 to 12,000	3.0 to 4.0
Very poor	< 7,000	< 2.1		

The main application for pulse-velocity measurements is to assess the uniformity of concrete. The technique is well suited to this purpose because up to 100 readings per hour can be taken under good conditions (*60*). The method has been used to establish areas of deterioration in concrete dams and pavements and could also be useful in assessing those parts of a structure most badly damaged by fire (*33, 60, 61*). There have been few reports of its application to highway structures (*36, 50, 52*). In one study, pulse-velocity measurements were made over a period of two years on several older bridges that were deteriorating because of alkali-aggregate reaction (*52*). In some parts of the structure the pulse velocities increased and in others they decreased. The inability to secure consistent readings from year to year was attributed to changes in moisture conditions with consequent changes in the width of the cracks or to exudations within the cracks. Another drawback is that highway structures are so heavily reinforced that the direct transmission mode is almost mandatory to produce reliable results. This

introduces the serious practical problem of gaining access to opposite faces of a member and ensuring that the transducers are aligned correctly. An attempt to use ultrasonic measurements on a bridge deck was unsuccessful (*36*) because much of the soffit was inaccessible (because of the presence of beams and diaphragms) and it was not possible to align the transducers such that the path length was known sufficiently accurately to detect defects or changes in the quality of the concrete. These examples do not mean that ultrasonic methods cannot be usefully applied to the evaluation of concrete in highway structures but they do illustrate some of the practical problems and emphasize the need to chose potential applications carefully.

An attempt has been made in England (*62*) to develop an ultrasonic technique for detecting fractures in prestressing strands. The method depends on having access to one end of the strand and a high-frequency pulse (1.5 MHz) is applied. A transducer is then moved along the surface of the concrete parallel to the strand. The major disadvantage of this approach is that a considerable portion of the energy is absorbed by the concrete such that the technique is only suitable for detecting fractures near the end of the beam. Work is in progress to extend the range of the method and to improve signal-processing methods.

Magnetic Methods

The main application of magnetic methods in the testing of concrete structures is in determining the position of reinforcement. Although not strictly a technique for detecting defects or deterioration, the fact that inadequate cover is often associated with corrosion-induced deterioration (*12*) means that locating the reinforcing bars can be an important component of a condition survey (*5, 12*).

There are several portable, battery-operated magnetic devices available that have been designed to detect the position of reinforcement and measure the depth of cover. These devices are known as cover meters or pachometers. The operating principle of the meters is that a battery generates a magnetic field between the two poles of a probe. The intensity of the magnetic field is proportional to the cube of the distance from the pole faces. When a magnetic material is present, a reinforcing bar for example, the magnetic field is distorted. The degree of distortion is a function of the diameter of the bar and its distance from the probe. The distortion is recorded on the meter, which is calibrated to indicate the distance between the probe and bar directly. Some meters also permit a correction for the effect of bar size. The performance of these devices is adversely affected at temperatures below 32° F (0° C) (*41*).

For accurate results, the axis of the probe must be parallel to the bars in the outermost layer of reinforcement. Conversely, where the orientation of the reinforcement is uncertain, it can be determined by rotating the probe on the concrete surface until a maximum disturbance (minimum cover reading) is obtained. When this occurs, the axis of the probe coincides with the direction of the reinforcement.

It is generally agreed that cover meters can measure cover to within 0.25 in. (6 mm) in the range of 0 to 3 in. (0 to 75 mm) although manufacturers have claimed greater accuracy. The instruments give satisfactory results in lightly reinforced members, such as bridge decks, but in heavily reinforced members, such as columns with spiral reinforcement, the effect of deeper bars cannot be eliminated and it is virtually impossible to obtain reliable results. Parallel bars also influence the meter reading if their spacing is less than two or three times the depth of cover (*23*). A further complication arises when some of the constituents of the concrete are magnetic because the measured cover is reduced. Some pozzolans, especially fly ashes, contain magnetic particles and many concrete sands contain particles of magnetite. In such cases, calibration curves must be established by exposing the reinforcement in some locations and comparing the recorded and actual values of concrete cover.

A rolling pachometer capable of gathering data at a rate of about twenty times that of hand-operated equipment has been developed for use on horizontal surfaces (*63*). The essential components of the machine are a hand-held cover meter with additional electronic components and a strip recorder, all of which are mounted on a small cart. The equipment is pushed along at a constant speed of 1 mph (1.6 km/h). Field experience has been variable; in some cases it has been found to be rugged and reliable and in others to be expensive and have only limited accuracy.

Magnetic methods have also been applied to solving the difficult problem of developing a practical nondestructive method of detecting loss of section or fracture of prestressing steel, primarily in beams. An extensive study of this problem was undertaken at the Southwest Research Institute under the sponsorship of the Federal Highway Administration (*45*). Several methods were investigated and preliminary studies indicated that magnetic field methods showed promise for detecting deterioration on strand in concrete and strand in a steel duct such that the method had potential application to both pretensioned and post-tensioned concrete structures.

The equipment that was developed involved the application of a steady-state magnetic field to the beam by means of a large, dc-excited electromagnet and scanning along the beam with a magnetic field sensor. As the sensor was moved parallel to the strands, perturbations could be detected and related to defects in the beam. The equipment had a good signal-to-noise ratio and could detect a fracture where the separation between the ends of the bar was as small as 0.01 in. (0.25 mm).

Following laboratory studies, a prototype unit was constructed for evaluation in the field. The electromagnet and sensor were mounted on a cart, which ran on a track hung from the bottom flange of I-section prestressed beams. The power supply, drive controls, and readout were all positioned at ground level. A field trial was undertaken in Utah. This involved substantial logistical problems because the cart, electromagnet, and sensor weighed 260 lb (118 kg) and had to be suspended beneath the beams. The results were encouraging in that perturbations were apparent, but it was difficult to relate the signal responses to physical defects. Signals from steel close to the surface, such as stirrups, tended to mask responses from steel deeper in the member. It was concluded that the full potential capability of magnetic methods could only be realized by developing better techniques for discriminating between signatures from deterioration and signatures from normal reinforcement details. A second research study was undertaken (*64*) to upgrade the prototype unit by using multiple sensors and improved signal acquisition and processing capabilities. The equipment has been found capable of detecting breaks in strand but has not been developed to the stage where it is suitable for routine operational

use. There are still some difficulties with signal recognition caused by stirrups and hardware items, such as chairs and tie wires. There is also the logistical problem of access, especially near piers. However, the method is promising and a more extensive evaluation of the capabilities of the equipment under field conditions is needed.

Electrical Methods

Electrical methods used in the field are currently limited to resistance and potential measurements. High-frequency capacitance measurements for determining moisture content (*11*) and polarization studies for measuring corrosion rates (*65, 66*) have been made in the laboratory but equipment suitable for field use has not yet been developed.

One of the first applications of electrical resistance testing was the development in California of a method of measuring the permeability of bridge deck seal coats (*67*). The method assumes that where a dielectric material is used to seal the surface of concrete, its electrical resistance is a measure of its watertightness. The procedure involves measuring the resistance between the reinforcing steel and a sponge on the concrete surface and has been published as a standard test method (ASTM D 3633). The method can be applied to any element with a nonconductive seal coat provided that it does not contain epoxy-coated reinforcement. Attempts to apply the test procedure to measure the effectiveness of membranes on asphalt-coated decks have met with limited success because of variations in pavement porosity and moisture conditions. Often, the presence of moisture in the bituminous surfacing results in a short circuit through a deck drain or steel expansion device so that the membrane cannot be evaluated (*12*).

More recently, interest in determining the resistivity of concrete has increased because resistivity is one of the factors that controls the rate of corrosion of steel in concrete. As the resistivity increases, the corrosion currents decrease such that corrosion is not of practical significance in dry concrete. It is most important to note that all measurements of the resistance of concrete must be made with an ac meter in order to eliminate polarization effects. To be precise, such measurements should be termed impedance rather than resistance because of the capacitance effects of the concrete.

Resistivity is normally measured by the four-electrode method (*65, 68*) common in geophysical testing. Four contact points are placed in the concrete (typically 1/4-in. or 6-mm deep), equally spaced in a straight line as illustrated in Figure 5. An alternating current is passed between the outer electrodes. The resulting potential difference between the inner electrodes is measured and the resistivity of the concrete calculated from the expression:

$$\rho = \frac{2\,\pi\,a\,E}{I} \qquad (5)$$

where

ρ = resistivity (in the same units as a),
a = electrode spacing,
E = potential drop between P_1 and P_2, and
I = current flow between C_1 and C_2.

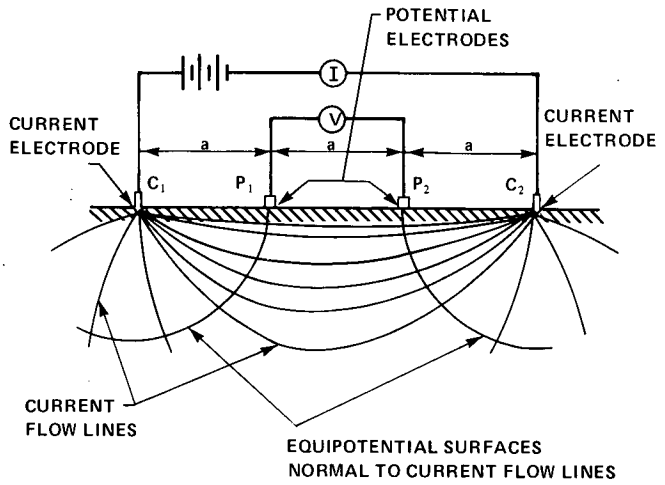

FIGURE 5 Four-electrode method of resistivity measurement.

The mathematical derivation of Eq. 5 is given in Appendix C. In theory, the current flow extends to an infinite depth but the intensity decreases quite rapidly with depth. For practical purposes, the penetration of the applied current can be considered equal to the spacing between the electrodes. When the electrode spacing is small, almost all the current will flow through the surface layers. If the electrodes are inserted deeper, or the spacing is increased, the effect of the reinforcement becomes significant. The major disadvantage of the method is that the value of resistivity measured is dominated by the resistance of the concrete near the surface whereas it is the resistivity of the concrete at the level of the reinforcement that is of primary interest.

Moist concrete usually has a resistivity of the order of 10^4 ohm-cm whereas oven-dried concrete has a resistivity of about 10^{11} ohm-cm (*65*). The resistivity associated with corrosion activity is somewhat uncertain. Observations on marine structures in California indicated that when the resistivity exceeded 60,000 ohm-cm no corrosion occurred but corrosion was detected below 60,000 ohm-cm (*69*). Work in the United Kingdom has indicated that corrosion only occurs at much lower values of resistivity and it was suggested that (*65*):

if the resistivity exceeds 12,000 ohm-cm, corrosion is unlikely;
if the resistivity is in the range 5,000 to 12,000 ohm-cm, corrosion is probable;
if the resistivity is less than 5,000 ohm-cm, corrosion is almost certain.

The figures assume that the concrete contains sufficient chloride ion to initiate the corrosion reactions. Other work in the United Kingdom (*70*) has suggested that corrosion is unlikely to occur in concrete with a resistivity in excess of 20,000 ohm-cm and that resistivities in the range 5,000 to 10,000 ohm-cm are needed to support corrosion activity. The reasons for the different limits are not clear. Two of the suggested ranges (*65, 70*) are based on the four-electrode method but the method of determining the resistivity in the California study was not reported. Further work is needed before resistivity measurements can be used as a reliable indicator of whether corrosive conditions exist in a bridge but the method does provide useful information to supplement potential measurements.

Consideration has been given to using the electrical resistance of steel as a method of determining the corrosion or fracture of reinforcement. Although it was once thought that the method showed promise for detecting a reduction in the cross-sectional area of bars, the spread of the data was so large as to make the method impractical. Using a simplified model, it was shown (*45*) that the presence of a 3-in. (75-mm) long section with only 6 percent of the cross section remaining increases the total resistance of a 40-ft (12.2-m) bar by only 10 percent. Furthermore a 10 percent variation could result from dimensional tolerances alone. This illustrates clearly that resistance measurements on steel bars are insensitive to reduction in cross section when access is available only to the ends of the beam. Furthermore, it is rarely practical to drill in order to make an electrical connection to the steel at frequent intervals.

When steel corrodes in concrete, a potential difference exists between the anodic half-cell areas and the cathodic half-cell areas along the reinforcing bar. The potential of the corrosion half cells can be measured by comparison with a standard reference cell, which has a known, constant value. A copper-copper sulfate (CSE) cell is normally used in field work because it is rugged, inexpensive, and reliable. The potential difference between the steel reinforcement and the reference cell is compared by connecting the two through a high-impedance voltmeter. This is done by connecting one lead of the voltmeter to the reinforcing steel. The other lead is connected to the reference cell, enabling electrode potentials to be measured at any desired location by moving the half cell over the concrete surface in an orderly manner. The cell can be used vertically downwards, horizontally, or vertically upwards provided that the copper sulfate solution is in contact with the porous plug and the copper rod in the cell at all times.

The reinforcement in each component of a structure is usually in good electrical contact so that only one electrical connection to the reinforcement is needed. It is, however, necessary to check that a good connection has been made and that the reinforcing steel is continuous by measuring the resistance to the reinforcing steel at another location or to exposed metal fixtures that are connected to the reinforcing steel. A separate connection is required wherever the steel is not continuous as, for example, in sections of a bridge deck separated by expansion joints.

Convention dictates that the potentials of steel relative to the copper-copper sulfate electrode be reported as negative values. The reason for this is that iron is more negative than copper in the electrochemical series.

On bridge decks that have a seal coat or membrane, the seal or membrane must be punctured at the point of measurement. Readings can sometimes be taken through an asphalt wearing course, but unless it can be shown that good electrical contact is made through the asphalt, it is good practice to drill holes through the wearing course to the concrete surface. If the asphalt contains significant amounts of moisture, or if the test is performed on concrete surfaces that are wet or under water, potential measurements will detect the presence of corrosion activity but not necessarily its location. A full description of the equipment and test procedures has been published by ASTM (ASTM C 876). Some of the practical considerations in applying the test procedure are discussed in references *5* and *12*.

It is generally agreed that the half-cell potential measurements can be interpreted as follows:

less negative than −0.20 V (CSE) greater than 90% probability of no corrosion,

between −0.20 and −0.35 V (CSE) corrosion activity is uncertain,

more negative than −0.35 V (CSE) greater than 90% probability that corrosion is occurring.

If positive readings are obtained, it generally indicates that there is insufficient moisture in the concrete and the readings should not be considered valid.

It is important to recognize the half-cell measurements indicate the potential for corrosion at the time of measurement, but give no information about the rate of corrosion. Corrosion rates of steel in highway structures are primarily controlled by the resistivity of the concrete and the availability of oxygen at the steel surface. Consequently, it is possible to have high potential measurements but low corrosion rates. It should also be recognized that, because corrosion is an electrochemical process and therefore temperature dependent, potential measurements will change with the time of year. These disadvantages are counterbalanced by the fact that the test is the only nondestructive test available that is a direct measure of corrosion activity. The technique is straightforward to carry out, relatively quick, and the results can be used to indicate which sections of a structure are likely to be corroding and which are not.

Chemical Methods

The principal chemical test methods applicable to concrete structures are those used to determine depth of carbonation and chloride ion content. Both methods are used to establish if the passivity of the reinforcement might have been destroyed.

The depth of carbonation can be measured by exposing a fresh concrete surface to a 2 percent solution of phenolphthalein in ethanol (*65*). Phenolphthalein is a pH indicator with a color change about pH 10. The magenta areas represent uncarbonated concrete and the colorless areas, carbonated concrete. Because of the presence of porous aggregates, voids, and cracks, the carbonation only approximates a straight line parallel to the concrete surface. A fresh concrete surface can be exposed by breaking off a piece of concrete with a hammer and chisel or by taking a core and breaking it in the laboratory.

The chloride-ion content of concrete is usually measured in the laboratory using a wet chemical method of analysis. However, the method of sampling affects field operations and there have also been attempts to measure the chloride content in-situ.

The simplest method of sampling for making chloride ion measurements is to take cores. The cores are sectioned in the laboratory and the sections of interest are pulverized and analyzed. To eliminate sawing and pulverizing in the laboratory, a percussion drill (sometimes called a rotary hammer) can be used to obtain a pulverized sample in the field (*71*). If it is desired to sample from several horizons, a hole is drilled until the required depth is reached, and the pulverized material is collected and placed in a sealed container. The hole is then thoroughly cleaned out with a vacuum cleaner before drilling for the next sample. A vacuum drilling method of obtaining powder samples for chloride-ion analysis was developed in Kansas (*72*). Sometimes it is only of interest to measure the chloride content

at the level of the reinforcement. When this is the case, the depth of cover is first measured with a cover meter. A hole is drilled to the level of the reinforcement, cleaned out, and the sample collected as already described. Combination core bits with a carbide-tipped starter bit have been found to increase the pulverizing action. In any event, all samples must be checked to be sure that they completely pass a 300-mm (No. 50) screen before testing. In some cases, a short period of pulverizing in the laboratory may be necessary.

Although pulverizing in the field offers the advantages of speed and economy, considerable care is required to prevent contamination of the samples. A common source of contamination is abrasion by the drill bit on the sides of the hole, especially near the surface where chloride levels are highest. This problem can be overcome by using a progressively smaller drill diameter for each successive sample taken. Guidance in selecting the location and number of samples is given in reference *12*.

The Kansas DOT developed a method for measuring the chloride content of bridge decks in-situ (*72*). The procedure involved drilling a 3/4-in. (19-mm) diameter hole in the deck to a predetermined depth. The hole was then filled with a borate-nitrate solution and a chloride-specific electrode inserted. After 90 seconds the potential across the electrode was measured and the chloride ion content determined from a calibration curve. The method had the advantages of speed and minimal damage to one deck but was discontinued because of the frequency with which electrodes were being broken in the field. It has not been widely used by other agencies, possibly because of the level of expertise required to conduct the test and the need to calibrate the electrode to known chloride concentrations.

Nuclear Methods

The chief application for nuclear methods is in the measurement of moisture content in-situ by neutron absorption and scattering techniques. The prime reason for measuring moisture content is to determine if corrosion of embedded reinforcement may occur.

An investigation was undertaken jointly by the Southwest Research Institute and the Portland Cement Association to determine the feasibility of using nuclear magnetic resonance (NMR) methods for nondestructively measuring moisture contents in concrete (*73*). A prototype piece of equipment was designed and fabricated. Laboratory testing proved the method to be feasible and initial field trials showed the equipment capable of providing reliable moisture content information as a function of depth. The equipment was modified to extend its operating range and further field trials were undertaken. The equipment is not available commercially. Although the method is feasible, the equipment is expensive, heavy, slow, and requires skilled operators. These drawbacks, together with the fact that direct measurement of the rate of corrosion would be more useful to the practicing engineer, mean that the technique is more likely to remain a research tool than become an operational procedure.

The effect of chlorine atoms on neutron beams was the basis of a technique for in-situ chloride determination developed by Columbia Scientific Industries under a pooled Highway Planning and Research Program contract (*74*). A laboratory evaluation showed a dual measurement method based on neutron bombardment to be feasible for measuring chloride ion content below the surface and without removing any of the concrete. The equipment, which was very large, was mounted on a self-contained vehicle and field trials were undertaken in Texas. Although it was claimed that six to ten determinations could be performed in one hour, only the prototype unit was ever constructed. An offer in 1981 to rent the equipment at a cost of $5000 per 5-day week was not taken up by any state. The main reasons for this were its high cost, limited accuracy, and logistical problems of moving from site to site as the vehicle could only travel very slowly. Furthermore, the equipment was conceived to permit on-the-spot decisions regarding bridge deck repairs but few agencies have used chloride-ion content to determine the extent of chloride removal or the method of repair.

Thermography

Infrared thermography has been found to be capable of detecting delaminations in bridge decks (*38, 75*). It can also be used on other concrete elements, such as columns, provided that they are exposed directly to the sun. The method works on the principle that as the concrete heats and cools, there is a substantial thermal gradient within the concrete because concrete is a poor conductor of heat. Any discontinuity, such as a delamination, parallel to the surface interrupts the heat transfer through the concrete. This means that, in periods of heating, the surface temperature of delaminations is higher than the surrounding concrete. At night, when there is usually a loss of heat from the concrete to the surrounding air, the surface of the delamination is cooler than the average temperature of the solid concrete. The time and the power required to heat concrete to establish the necessary thermal gradients mean that artificial sources of heat cannot be used.

The differences in surface temperature can be measured using sensitive infrared detection systems. The essential components of such a system are an infrared scanner, control unit, battery pack, and display screen. The images can be recorded on photographic plates or videotape. Either color or black-and-white images can be used but the black and white is much easier to interpret and is better suited to structural applications. Not only was it found possible to identify areas of delamination but differences in surface temperature were found to correlate with the depth of the delamination below the surface. In other words, there was a larger temperature difference associated with shallow delaminations than with deeper ones.

A number of operational modes have been used ranging from a hand-held scanner at deck level (*75*) to boom trucks and helicopters (*38*). A truck-mounted platform 13 to 20 ft (4 to 6 m) above the deck has been found to give the best results with respect to definition, accuracy, and speed (*36*). This configuration also permits a lane width to be scanned at a single pass and this is very convenient in the field. When used in this way, thermography is at least as accurate as the Delamtect (*38, 76*) but not as good as a chain drag.

Under ideal conditions of summer sun with no cloud or wind, differences in surface temperature as large as 4.5° C (8.1° F) have been recorded between delaminated and solid sections of a deck. The temperature differential is reduced by wind, cloud cover, and high humidity. Thermography cannot be used when moisture is present on the deck surface because of its high emissivity.

Infrared thermography can also be used to identify delamination and scaling in asphalt-covered deck slabs although the technique is much more sensitive to weather conditions than when applied to bare decks. The maximum temperature difference of a 3.1-in. (80-mm) bituminous surfacing over a delaminated area and sound concrete was measured as 2° C (3.6° F) and the "window" during which delaminations could be detected extended from 11:30 am to 6:00 pm (*36*).

In addition to constraints imposed by weather, the main disadvantage of thermography is that while a positive result is valid, a negative result may mean that the deck is free from deterioration or that it contains deterioration that could not be detected under the conditions prevailing at the time of test. This problem is expected to become less serious as more experience is gained in predicting the conditions under which thermography will identify deterioration. A further drawback is that it is difficult to produce scaled hard copy showing the areas of deterioration on a plan of the deck. Nevertheless, the method does have considerable promise as a rapid-screening tool on both asphalt-covered and exposed concrete deck slabs to assess priorities and determine if a more detailed investigation is required. The equipment for infrared thermography is readily available on a rental basis and its use is expected to increase.

Radar

Investigations into the use of ground-penetrating radar for detecting deterioration in pavements and concrete bridge decks began in the mid 1970s (*77, 78*). These investigations were prompted by the development in the 1960s of low-power, high-frequency pulsed radar, which offered the resolution necessary to detect small flaws in concrete. The pulses are of extremely short duration, approximately one nanosecond.

The equipment consists of a monostatic antenna, transmitter, receiver, and oscilloscope as shown in Figure 6. Pulses of radio frequency energy are directed into the deck, a portion is reflected from any interface, and the output is displayed on an oscilloscope. An interface is any discontinuity or differing dielectric, such as air to asphalt, asphalt to concrete, or cracks in concrete (*79*). A permanent record can be stored on film or analog magnetic tape. The unit is normally mounted in a vehicle and the data are collected as the vehicle moves slowly across the deck. The antenna travels 6 to 8 in. (150 to 200 mm) above the deck surface and has a "footprint" of approximately 1 ft^2 (0.09 m^2).

A number of studies have been carried out on both bare concrete and asphalt-covered bridge decks (*36, 77, 80, 81*). In all cases the radar was found to be capable of identifying anomalous areas in the deck. The practical problem was analyzing the large amount of data that were collected and relating the different radar signatures to physical distress. Lack of experience in interpreting the data initially led to a large number of false results.

An important benefit of radar on asphalt-covered decks is the ability to measure the thickness of the asphalt surfacing. The echos produced from the pavement surface and the asphalt-concrete interface are very distinct such that the thickness can be determined accurately. It also has the potential for examining the condition of the top flange of box beams that are otherwise inaccessible (*81*).

The use of radar has considerable potential for the automatic processing of the output signal (*36, 80*). It is anticipated that it will eventually be possible to produce a site plan of a structure identifying the extent and type of deterioration directly from the magnetic tape and a knowledge of the deck geometry. The biggest advantage of radar over thermography is that it is almost independent of weather conditions. The only environmental influence is that radar does not work effectively if the deck surface is wet or if there is significant moisture in a bituminous surfacing because of attenuation of the signal. On the negative side, several passes of the deck are required when a single antenna is used

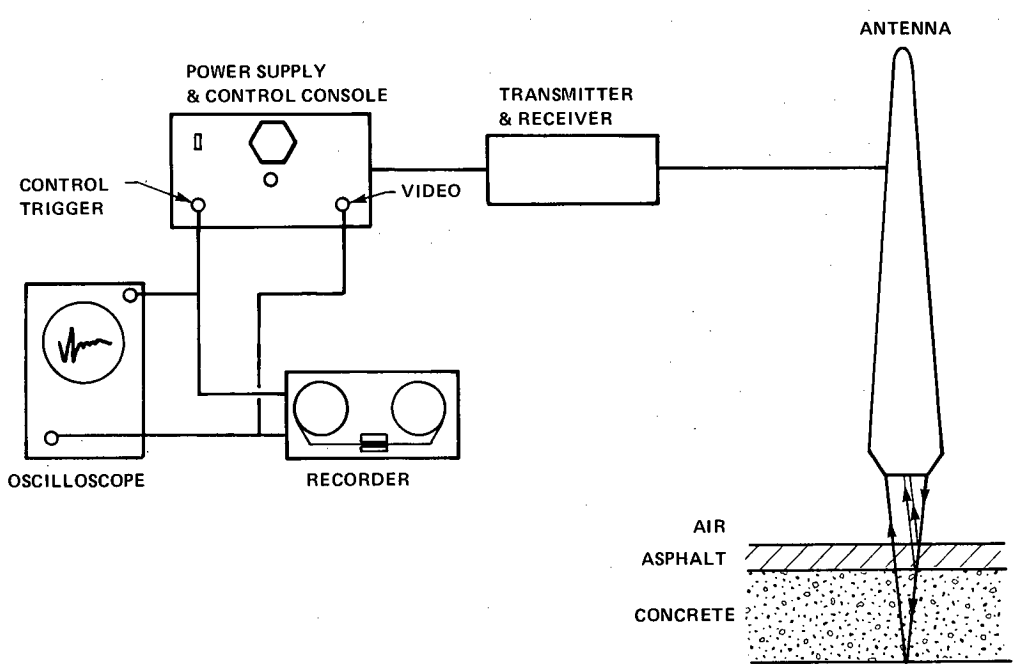

FIGURE 6 Elements of a radar system.

in order to survey the surface adequately. There are also few systems available and this may inhibit its widespread use over the short term.

Radiography

There have been numerous attempts dating back to 1949 to apply X rays and gamma radiography for the nondestructive examination of concrete, especially in Europe (*82–86*). As discussed in Chapter 2, X rays and gamma rays differ only in their origin. This difference has considerable practical significance because X-ray equipment has a high initial cost, produces very high voltages, and is not portable with the result that X rays offer little scope for use in the field (*23, 87*).

The principle involved in radiographic methods is that as the radiation passes through a material of variable density, more radiation is absorbed by the denser parts of the material than the less dense parts. Most applications of radiographic techniques involve the transmission of wave energy rather than the reflection and refraction methods common with lower frequency methods. However, back-scatter techniques can be used where only one face of a member is accessible; although, as with the indirect method of ultrasonic testing, this is much less satisfactory than direct transmission.

Two experimental techniques, radiography and radiometry, are employed (*11*). In radiography, the emerging radiation is detected by a photographic emulsion and variations in the density of the exposed film reflect the internal structure of the material under examination. In radiometry, variations in the gamma intensity are detected by radiation detectors, such as Geiger or scintillation counters and measured by associated electronic apparatus.

Radiography with gamma rays has been investigated for detecting variations in consolidation in members up to about 18-in. (450-mm) thick, locating reinforcement, measuring the extent of corrosion, and assessing the quality of grouting in prestressing ducts (*11, 82–84*). It is also used in Eastern Europe and the Soviet Union for quality-control testing (*11, 27*). At thicknesses greater than about 18 in. (450 mm), the long exposure needed makes the process uneconomical (*86*). It should be noted that concrete is often used as a shielding material against high-energy X rays and gamma rays because it is an effective absorber. This characteristic clearly limits the application of radiography to relatively thin members.

Many of the methods of measuring in-situ density involve drilling holes in the concrete and placing the radioactive source or the detector in the hole (*23, 88*). This technique can be used to detect honeycombing or voids but cores are also necessary for calibration purposes if density is to be measured in absolute terms. Although it may be justifiable in determining if a defective component in a structure should be replaced at the time of construction, or for checking remedial measures, there are few occasions when such testing is warranted on highway structures already in service.

The main application for gamma radiography is for detecting defects in prestressed concrete structures. Laboratory work in England (*84*) showed that radiography could detect voids in grout as small as 3/16 in. (5 mm) in concrete beams 5-in. (125-mm) thick. It has also been used in the field on sections up to 16-in. (400-mm) thick (*21*). Most of the experience with radiography in the field has been in France where it has been used since 1968 to locate the position of prestressing cables, detect defects in cables, and examine the quality of grout (*89*). More recently, the equipment has been developed into a system that makes possible the detailed inspection of cables in box, I-section, and slab bridges (*90*). The system is known as SCORPION (from radioSCOpie par Rayonnement Pour l'Inspection des Ouvrages en betoN, which can be translated as radiation radiography for the inspection of concrete structures). It consists of a radioactive source, a detector, and a remote command module as illustrated in Figure 7. The detector includes a filter, a converter (which forms an optical replica of the incident radiation), a mirror, and a low-light-level television camera. The source and detector are mounted on moveable platforms, which can be operated by remote control from the command module.

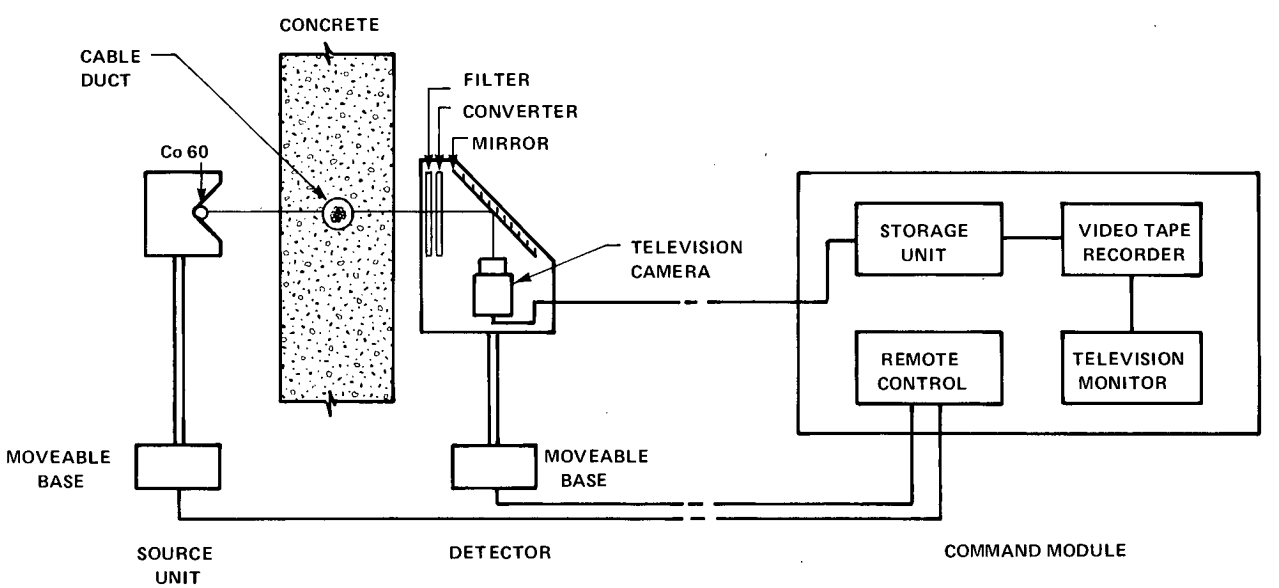

FIGURE 7 Elements of a SCORPION system.

The command module also includes a television monitor, storage unit, and video-tape recorder. A prototype unit was constructed in 1979 and has been found capable of examining concrete thickness up to 18 in. (450 mm) in field trials. In September 1984, construction began on another unit, SCORPION 2, that will utilize a linear accelerator as the source of radiation, giving the unit the capability to examine concrete up to 4-ft (1.2-m) thick.

The developments in France in inspecting prestressed concrete have not been publicized in North America. Although the system appears viable, independent confirmation of its capabilities is needed. Radiographic techniques are well suited to detecting voids in grout and cables that are broken or out of position. However, it should be recognized that small amounts of corrosion will not be detected.

Air Permeability

The main application of air permeability measurements is to obtain an in-situ assessment of the resistance of concrete to carbonation and to penetration of aggressive ions.

A field method of measuring air permeability has been investigated in Japan (*91*). The procedure consists of drilling a 1/4-in. (5-mm) diameter, 1 1/2-in. (40-mm) deep hole into the concrete. The hole is sealed with a rubber stopper and an injection needle is inserted through the stopper and connected by a hose to a mercury manometer and vacuum pump. The air in the hole is evacuated and the rate at which the manometer recovers is a measure of the permeability of the concrete. The technique is still considered experimental.

A different technique has been developed in Denmark (*92*). A 3/4-in. (18-mm) hole is cut to a predetermined depth at an angle of 8° to the surface. A pressure sensor with sealing ring is inserted to the end of the hole and low pressure air is applied to the surface of the concrete directly over the sensor. The response of the sensor is recorded. A supplementary test has been devised to ensure that the concrete is not cracked because the existence of cracks would offer a preferred path to the flow of air. Further development is continuing.

A different application of air permeability measurements has been made in the United Kingdom where they have been used as a method of assessing the quality of grout in post-tensioned cable ducts (*21*). The method involves drilling holes into the top of the duct. The duct is then evacuated at one hole and the amount of vacuum measured at the other holes in the same duct to determine whether voids are continuous along the duct. The volume of any voids present was measured by connecting the evacuated voids to a water gauge consisting of a narrow plastic tube dipping in water. The height to which the water was drawn up the tube was measured and the volume of voids calculated. Although the method is subject to errors because of leakage into the duct and possible presence of undetected voids, it does give an indication of the quality of grouting in ducts accessible by drilling.

A refinement of the method was to measure the rate at which air could leak out of the ducts by pressurizing them with nitrogen gas and measuring the input flow rate required to maintain a pressure of 2.5 psi (17 kPa) above atmospheric pressure. When high flow rates were measured, a soap solution was applied to the exterior of the concrete so that points of leakage could be detected.

Comparison of Test Methods

A general summary of the capabilities of the various test methods to detect different forms of defects or deterioration is given in Table 3. The table permits comparison among the methods and can be used in planning an investigation. An indication of the costs of many of the techniques listed in Table 3 is given in references *36* and *93*.

Some of the techniques for detecting corrosion of embedded steel measure corrosion activity or corrosion-induced distress directly, whereas others measure one or more of the properties of concrete that are a factor in causing corrosion. A classification of the techniques that may be used in examining reinforced concrete structures for corrosion is given in Table 4.

Table 3 indicates that a good visual survey, complemented by a judicious selection of techniques to investigate further the defects that have been identified, is a sound approach. However, the major weakness of a visual survey is the inability to detect hidden defects. Detecting corrosion of embedded reinforcement is sufficiently difficult that a number of techniques, such as concrete cover, half-cell potential, and chloride-ion content, are normally performed. Under such circumstances the techniques should be considered complementary rather than competitive options. The nature of chemical attack and factors in the quality of the concrete that may have contributed to scaling, wear, or abrasion damage are best determined by removing cores and examining them in the laboratory.

TABLE 3

CAPABILITY OF INVESTIGATING TECHNIQUES FOR DETECTING DEFECTS IN CONCRETE STRUCTURES AND FIELD USE

	Capability of Defect Detection[a]					
Technique	Cracking	Scaling	Corrosion	Wear and Abrasion	Chemical Attack	Voids in Grout
Visual	G	G	P	G	F	N
Hardness	N	N	P	N	P	N
Sonic	F	N	G	N	N	N
Ultrasonic	G	N	F	N	P	N
Magnetic	N	N	F	N	N	N
Electrical	N	N	G	N	N	N
Chemical	N	N	G	N	N	N
Nuclear	N	N	F	N	N	N
Thermography	N	G[b]	G	N	N	N
Radar	N	G[b]	G	N	N	N
Radiography	F	N	F	N	N	F
Air Permeability	N	N	F	N	N	F

[a]G = good; F = Fair; P = Poor; N = Not suitable

[b]Beneath bituminous surfacings.

TABLE 4

CLASSIFICATION OF TECHNIQUES FOR EXAMINING REINFORCED CONCRETE STRUCTURES FOR CORROSION (ADAPTED FROM REFERENCE *99*)

Technique	Direct	Indirect	Non-Destructive	Semi-Destructive	Destructive	Corrosion		
						Rate	Defect	Causes
Visual	X		X				X	
Weight Loss	X				X	X		
Pit Depth	X				X	X		
Half-Cell Potential	X		X				X	
Carbonation		X			X			X
Covermeter		X	X					X
Chloride Analysis		X		X				X
Resistivity		X	X					X
Moisture Content		X			X			X
Water Absorption		X			X			X
Concrete Strength		X			X			X
Concrete Permeability		X			X			X
Delamination		X	X				X	
Ultrasonic Methods		X	X					X
Hardness Methods		X	X	X				X
Radiography		X	X					X
Coring		X			X			X

Table 3 shows that only radiography and air-permeability measurements can be used to detect voids in grouted cable ducts and neither method is ranked better than fair. None of the methods are well suited to detecting corrosion on cables contained in ducts.

LABORATORY PROCEDURES

Although most of the test methods described for use in the field could also be used in the laboratory, there is little incentive to do so. Strength can be measured directly and other tests are available to evaluate the quality of the concrete that provide much more detailed information than the indirect methods that must be used in the field. In contrast with steel and timber, samples (usually cores) can be taken from concrete structures and the concrete replaced easily and inexpensively. Consequently, it is common practice to take cores from both sound and deteriorated parts of concrete structures for laboratory testing. Guidance on selecting the location and the number of cores for testing is given in references *5* and *12*.

Most of the laboratory test methods for concrete are relatively straightforward and are listed in Table 5.

Petrographic Analysis

A petrographic analysis is a detailed examination of the material, composition, and structure of the concrete. It may include identification of the mineral aggregates, examination of the paste-aggregate interface, and an assessment of the structure and integrity of the paste itself. A petrographic analysis can be extremely useful in identifying such mechanisms of deterioration as freeze-thaw, sulfate attack, and alkali-aggregate reactivity. It can also be used to assess the severity of fire damage. Specimens can be prepared in a number of ways, including polished surfaces, etched surfaces, and thin sections. The work is very specialized and is normally undertaken only by a qualified petrographer.

Physical Test Methods

The strength of concrete can be measured directly by testing cores to failure in compression. The elastic modulus can be determined from the linear portion of the stress-strain curve. Alternatively, both the elastic modulus and flexural strength can be predicted with reasonable accuracy from a knowledge of the compressive strength. Tensile strength can be measured on cores by the indirect tension or Brazilian test. It is usual to test cores in the same moisture condition as they are in service, especially when carrying out structural evaluations. Cores should preferably have a diameter three times the maximum aggregate size and must be at least twice the maximum nominal size of the coarse aggregate. These ratios apply to the actual diameter of the concrete core and not the nominal diameter of the core bit. Cores for compressive strength measurements should have a length-to-diameter ratio (L/D) of 2.0. This is not always possible, especially in cores from decks but, in all cases, the L/D must not be less than 0.95 before capping and 1.0 after capping. The measured strength is then converted to the strength of a core having an L/D of 2.0 using the correction factors given in ASTM C 42. Strictly speaking, cores for strength testing should be free from reinforcement. Again, this is not always possible. Horizontal bars through the core tend to reduce the measured strength because they act as stress concentrations. However, there is no agreement on the correction factors that should be applied for various configurations of reinforcement

to give the equivalent strength of plain concrete. Cores that are cracked should not be tested for strength.

The density of absorption of concrete samples can be measured with a minimum of effort and expense. The main benefit of such measurements is that they give an indication of the variability of the concrete within the same structure.

A measurement of the air-void system of the concrete by either the linear-traverse or modified-point-count method is the best method of determining its resistance to freezing and thawing. Concrete is considered to have a satisfactory air-void system when the spacing factor is no greater than 0.008 in. (0.20 mm), the specific surface is greater than 600 in.^{2}/in.^{3} (24 mm^2/mm^3), and the number of air voids per inch of traverse is more than twice the numerical value of the percentage of air in the concrete (*94*).

A relatively new test method is the rapid measurement of chloride permeability (*95*). The test consists of monitoring the amount of electrical current passed through a 2-in. (50-mm) long core when one end of the core is immersed in sodium chloride solution and a potential difference of 60 V dc is maintained across the specimen for 6 hr. The total charge passed, in coulombs, is related to chloride permeability. Although the technique is ideally suited to the evaluation of sealants and modifiers to decrease the permeability of concrete, it is also useful as a means of assessing the overall quality of concrete and measuring the resistance of plain concretes to penetration by chloride ions.

Field trials of the test procedure were conducted on two bridges in Wisconsin in 1980 (*96*). The results showed significant variations in concrete quality within each bridge although additional work remains to be done to determine the effects of clear cover, temperature, and use of materials other than plain portland cement concrete. The time acquired for testing (one traffic lane closed continuously for five days to conduct four in-situ tests) is a major disadvantage in the use of the equipment in the field.

Efforts to determine the extent of corrosion by weight loss or pit-depth measurements (*65*) are not very useful and can be very misleading because the corrosion of steel in concrete is not uniform. The absence of rust on the reinforcing steel in a core is not a valid indication of the absence of corrosion in the structure because the core may have been taken from a cathodic area.

TABLE 5

STANDARD ASTM AND AASHTO TEST METHODS FOR CONCRETE FOR USE IN THE LABORATORY

Designation[a]	Title
C 39 T 22	Test Method for Compressive Strength of Cylindrical Concrete Specimens
C 85 T 178	Test Method for Cement Content of Hardened Portland Cement Concrete
C 174 T 148	Method of Measuring Length of Drilled Concrete Cores
C 457	Practice for Microscopical Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete
C 469	Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression
C 496	Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
C 617 T 231	Method of Capping Cylindrical Concrete Specimens
C 642	Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete
C 666 T 161	Test Method for Resistance of Concrete to Rapid Freezing and Thawing
C 856	Recommended Practice for Petrographic Examination of Hardened Concrete
T 259	Method of Test for Resistance of Concrete to Chloride Ion Penetration
T 260	Method of Sampling and Testing for Total Chloride Ion in Concrete and Concrete Raw Materials
T 277	Interim Method of Test for Rapid Determination of the Chloride Permeability of Concrete

[a]ASTM test methods are designated C.
AASHTO test methods are designated T.

Chemical Test Methods

The chemical test methods of greatest importance are for carbonation and chloride ion content. It is also possible to test for cement content but it is difficult to obtain reliable results. Furthermore, the most common requirement for cement content determinations is in disputes at the time of construction rather than in investigating existing structures.

The depth of carbonation is measured by indicator solutions using the method described in the field procedures section.

The chloride ion content is measured by a wet chemical process in one of two ways, depending on the method of extraction. The total chloride-ion content is measured by dissolving a powdered sample of the concrete in dilute nitric acid and conducting a potentiometric titration of the chloride ion with silver nitrate solution. The method is both reproducible and accurate. The soluble chloride-ion content is measured by using water to extract the chloride ion. The amount of chloride extracted depends on the time and temperature of extraction. Most agencies use a period of boiling of 5 minutes and then let the sample stand for 24 hours (*97*).

Although the chemical aspects of the test for chloride ion are straightforward, interpretation of the results is not. The total-chloride method measures all the free and the chemically bound chloride ion in the concrete. This means that the test result includes an unknown quantity of chloride ion that is not available to contribute to corrosion. The water-soluble test is thought to measure the free chloride ion and some of the chemically bound chlorides, and thus has no real advantage over the total-chloride method. The quantity of chloride ion that is sufficient to initiate corrosion is approximately 0.20 percent by weight of cement (or about 0.03 percent by weight for a typical concrete) when measured by nitric acid extraction on concrete samples taken at the level of the reinforcement. However, this quantity does not include chloride ions that are present in the mix ingredients and that cannot enter into the pure water solution. Further details and guidance in the interpretation of test results are given in reference (*12*).

CHAPTER FOUR

STEEL AND WROUGHT-IRON COMPONENTS

CHARACTERISTICS OF STEEL AND WROUGHT IRON

Steel is a versatile construction material that has been widely used in many highway structures and used in many forms; notably plate, rolled sections, cables, and, for a few old bridges, chains. Steel is a much more homogeneous material than either concrete or timber and is isotropic. It possesses very high compressive and tensile strength and is also strong in shear. As specifications have changed, the ultimate tensile strength of steel used in bridges has also been revised periodically over the years, typically being in the range of 50,000 psi (350 MPa) to over 80,000 psi (555 MPa). Yield strengths were normally not less than 30,000 psi (210 MPa) or half the tensile strength. The actual tensile strength of the steel in a specific structure depends on the grade, the component, and the specification current at the time of manufacture. Useful information on the strength requirements of both steel and wrought iron during the period 1873 to 1952 is given in reference *98*.

When used in thin sections, steel is vulnerable to buckling under compressive load and must be stiffened. The low-carbon and low-alloy steels are normally used in bridges and these are ductile. However, brittleness may occur because of heat treatment, welding, or as a consequence of metal fatigue.

Steel is elastic and conducts both electricity and heat. When exposed to high temperatures, such as those resulting from a fire, it is subject to loss of strength. The major disadvantages of unprotected carbon steel is its susceptibility to corrosion and hence the need to provide and maintain a means of corrosion protection.

To understand the properties of wrought iron, a brief explanation of the method of manufacture is necessary. Wrought iron is made by removing impurities from pig iron. As the impurities are removed in the furnace, the metal solidifies into a pasty metal closely intermixed with a considerable quantity of slag. The pasty balls are removed, hammered, and rolled into crude bars. The bars are then cut to length, bundled, heated, and rolled into sections. The rolling serves to weld the layers together and to elongate the slag into fibers. Consequently the finished wrought iron consists of slag fibers in a soft iron matrix. This means that wrought iron, like timber, is anisotropic with different properties along and across the grain.

The chief property of wrought iron is that it is easily worked. In forge-welding the slag acts as a flux and also tends to prevent grain growth, which would cause embrittlement. Some types of wrought iron, depending on the impurities present, are more resistant to atmospheric corrosion than mild steel. The fibrous nature of the material produces a light rust, which is less likely to progress to flaking and scaling than is rust on carbon steel (*4*).

Wrought iron has an ultimate tensile strength of about 50,000 psi (350 MPa) along the grain (i.e., in the direction of rolling). The strength across the grain is about 75 percent of its longitudinal strength. The modulus of elasticity is in the range 24 to 29 $\times$ 10^6 psi (170 to 200 GPa). Wrought iron is generally ductile, although this is affected by the method of manufacture. It is tough and resistant to impact.

In some early iron bridges, wrought iron was used in combination with cast iron (*4*). Wrought iron was used for the tensile members and cast iron for the compressive members by virtue of its high compressive strength of 60,000 psi (420 MPa) and up. However, the tensile strength of cast iron varies from 20,000 to 60,000 psi (140 to 420 MPa), depending on its composition. It is also very brittle and subject to checks and blowholes, which reduce its strength considerably.

COMMON DEFECTS AND TYPES OF DETERIORATION

The most common forms of defects and deterioration that occur in steel and wrought-iron components are described below. Photographs of typical examples of these defects are contained in references *4* and *7*.

Corrosion

Corrosion damage usually occurs sooner or later on all kinds of steel in bridges and increases with time. Unprotected steel, in the presence of oxygen and moisture and in the absence of contaminants, corrodes at approximately 0.008 in./yr (0.02 mm/yr). Thus, except over very long periods, this type of uniform corrosion on a surface does not have dramatic consequences. However, corrosion is aggravated by continuous wet conditions or exposure to aggressive ions, such as chlorides present in deicing salts or a marine environment or sulfides in industrial atmospheres. In these circumstances, steel is vulnerable to pitting or local corrosion, which may amount to approximately 0.012 in./yr (0.3 mm/yr) or more (*15*). Pitting corrosion can cause a serious reduction in load-carrying capacity and introduces a particular risk of fatigue failure. Although less widespread, other causes of corrosion of steel components are animal wastes, the spillage of farm fertilizers, the flux used in welding (if not neutralized), and direct contact with dissimilar metals (*99*).

Since the mid 1960s, a significant proportion of the structural steel components of highway bridges has been atmospheric corrosion resistant (ACR) steel or weathering steel, which does not require painting (except in the very worst exposure conditions). Corrosion protection is provided by the natural development of a tightly adhering oxide layer. The ideal conditions for the development of a stable oxide film are direct exposure to sunlight and intermittent wetting and drying in the absence of salts and other contaminants. Unfortunately, except for brief periods during construction, these conditions do not exist in most highway

bridges. Furthermore, the steel components are often exposed to surface run-off containing deicing chemicals. Frequently, ventilation is poor so that steel that becomes wet does not dry. These conditions have led to a much slower development of the oxide layer than initially envisioned and localized accelerated corrosion in those parts of the structure most vulnerable to corrosion, i.e., horizontal surfaces and areas near drains and leaking joints (*100–102*). The main difference between the corrosion of weathering steel and mild steel is that, rather than pitting corrosion, the corrosion products tend to be sheets or laminae of flaky material.

Cracking

The most common causes of cracking are fatigue and poor detailing practices that produce high stress concentrations (*4, 103*). Fatigue begins at fabrication flaws and proceeds by the growth of these flaws until a final failure mode, such as brittle fracture or buckling, occurs. The initial fabrication flaws may be large or small but in many cases are too small to be detected by eye or even by current nondestructive test methods (*104*). Cracks in the steel may range from hairline width to sufficient width to transmit light through the members. The variation in length and depth is of a similar magnitude. Cracks may also be present in welds because of poor welding techniques or the use of steel with poor weldability.

Bent or Twisted Members

Distortion of members occurs because of an overload condition, which could arise in a number of ways. The most common are excessive live loads, vehicle collisions, thermal strains resulting from frozen bearings or expansion joints, and fire. Increased stress may result from failure or yielding of adjacent components or from deterioration of the member itself.

FIELD PROCEDURES

There exists a much wider variety of test methods that can be used on steel components compared to concrete or timber because steel is more homogeneous and isotropic and, therefore, flaws and discontinuities are, in general, easier to recognize. Many of these procedures can be performed in the laboratory but are not easily done on the superstructure of a highway bridge, high above the ground and under adverse weather conditions. In fact, the ability to use equipment under difficult working conditions can be the most important criterion in deciding to use a particular test method in the field. The equipment and procedures that are described in this section are those that are considered to be most suitable for field use. Applicable test methods are given in Table 6.

TABLE 6

STANDARD ASTM TEST METHODS FOR STEEL COMPONENTS FOR USE IN THE FIELD

Designation	Title
B 499	Method for Measurement of Coating Thickness by the Magnetic Method: Nonmagnetic Coatings on Magnetic Basis Metals
D 1186	Methods for Nondestructive Measurement of Dry Film Thickness of Nonmagnetic Coatings Applied to a Ferrous Base
E 94	Recommended Practice for Radiographic Testing
E 114	Recommended Practice for Ultrasonic Pulse-Echo Straight-Beam Testing by the Contact Method
E 165	Practice for Liquid Penetrant Inspection
E 376	Recommended Practice for Measuring Coating Thickness by Magnetic Field or Eddy-Current (Electromagnetic) Test Methods
E 494	Recommended Practice for Measuring Ultrasonic Velocity in Materials
E 569	Practice for Acoustic Emission Monitoring of Structures During Controlled Stimulation
E 587	Practice for Ultrasonic Angle-Beam Examination by the Contact Method
E 709	Recommended Practice for Magnetic Particle Examination

Visual Inspection

Once the material in a structure has been recognized as steel or wrought iron, inspection procedures are similar. Where wrought iron cannot be identified from records, appearance, or structural form, two test methods are described elsewhere in this chapter.

The general principles of systematically inspecting each component, recording defects and deterioration, and photographing significant items that were described for concrete structures also apply to steel, wrought-iron, and timber structures.

Where the structure is painted or galvanized, it is usual to observe and record the condition of the protective coating, estimating the percentage of coating breakdown and the severity of any corrosion at breaks in the coating.

Rusted steel varies in color from dark red to dark brown. Initially the rust is fine grained but, as it progresses, it becomes flaky or scaly in character. The degree of rusting is usually assessed qualitatively using a scale such as light, moderate, and severe (*4*). The depth of any pitting and the size of any perforations should be measured and recorded. On weathering steel it is important to note the cohesion and porosity of the surface.

Those parts of any steel structure where moisture is trapped, particularly in association with salt, are the most vulnerable to corrosion. These include any component where debris, such as moist sand, can accumulate, but especially horizontal surfaces, connections, and steel in close proximity to leaking deck joints or the discharge point of deck drains. Other areas vulnerable to corrosion are mating and rubbing surfaces (where the critical areas are out of sight) and closed members, such as boxes and columns. Accumulation of water caused by leakage and condensation is a common occurrence that leads to potential corrosion problems in closed members. Inspection in closed and confined places should be carried out with good lighting and with suitable precautions to ensure that the atmosphere is nontoxic (*4*).

Any type of crack can be serious and the location and size of all cracks must be recorded. Paint and corrosion products can sometimes make the observation of cracks difficult; at other times the crack will break the paint causing rippling or rust staining, which aids detection. The detection of fatigue cracks is aided considerably by the fact that the location of the cracks is at least predictable. Fatigue is a consequence of the high stress range that occurs in steel structures under live load and the areas of high stress concentrations are known for a given bridge structure. Examples of such areas and details are: re-entrant corners, abrupt changes in plate width or thickness, a concentration of welds (especially at the toes), and insufficient bearing area for a support. A comprehensive handbook has been prepared that contains color photographs and line drawings where fatigue cracks have developed in bridge structures (*105*).

Careful observation is also required for members that are overstressed, deformed, or misaligned and also for missing or broken fasteners. Although missing fasteners are usually apparent, broken fasteners may be difficult to detect unless accompanied by movement or noise. Occasionally, broken fasteners can be detected by broken paint films between cover plates and framing members. Listening to the response of the structure to live load can also be useful in detecting defects such as frozen pins, rockers, and bearings or loose deck joints. Collision damage from vehicles can usually be identified from notches or cuts, misalignment, disturbances in the paint or surface oxide layer, or paint from the vehicle. Evidence of slight overload conditions is usually found in the form of fine cracks or ripples in a paint surface resulting from large strains. Such conditions are most common near connections where additional evidence may be found in the form of deformed bolts or rivets.

Simple Test Procedures

There are a number of relatively simple test procedures that can be used to assist in identifying the type of material or the grosser defects in steel and wrought-iron structures.

Where a strength evaluation of a metal structure is to be carried out, identification of the metal is necessary. Identification in rather general terms (for example, steel, wrought iron, or aluminum alloy) will establish the elastic modulus of the material. If hardness tests are combined with the identification, estimates of strength and possible chemical spot tests can be used to identify specific alloys or groups of alloys (*106*). However, many of the procedures are suited to the laboratory and the results of tests carried out in the field are not always as clear as desired. The chemical tests should not be performed on highly stressed areas of a member as adverse corrosion effects are possible. Spark tests provide a means of distinguishing between high- and low-carbon steels and wrought iron (*107, 108*). They should be made in dim light and tests should be made on known materials under the same conditions for comparison.

Wrought iron can also be identified by examining filed and lightly ground surfaces for slag inclusions. The direction of filing or grinding should not be parallel to the direction of rolling to avoid scratches that could be mistaken for slag inclusions (*108*). Care should be taken not to leave scratches transverse to the load direction, which may initiate a crack.

In painted structures, visual examination can be supplemented by taking film thickness measurements. The thickness gauge measures the strength of the magnetic field between a magnet in the gauge and the metal in the structure. It is calibrated to express the coating thickness directly and instruments are available for different ranges of thickness. The gauge can be used to measure the thickness of any nonmagnetic coating applied to a ferrous base. Paint and galvanizing thickness are the two most important applications in highway structures. The gauges are easy to use and are accurate to approximately 0.001 in. (0.025 mm), and a large number of readings can be taken in a short time. The use of a radiometric thickness gauge using beta or gamma rays has been reported in Europe (*6*). A scintillation counter receives the radiation and output pulses are transferred into a current proportional to the thickness of the coating. The device is reported to be suitable for measuring a wide range of coating thickness.

An alternative approach to measuring paint thickness is by microscopic observation of a small v-groove cut into the paint film (*93*). A commercial unit, the Tooke gauge, exists for this purpose. The number of paint layers and individual thicknesses can readily be determined. The equipment is inexpensive and easy to operate but has the disadvantage of making a cut in the paint film, which must be repaired.

Pin holes can be detected using battery-operated commercial instruments. These consist of an electrode and moist sponge that make contact with the paint surface and a second terminal that is attached to bare metal. When the moist sponge passes over a pinhole the electrical circuit is completed and an alarm sounds.

A pinging ball or a light hammer can be used to test for loose or broken rivets (*109*). A torque wrench will identify loose or broken bolts.

Where access can be obtained to opposite surfaces of a component, a micrometer can be used to measure section loss. In cases where there are heavy corrosion deposits, mechanical cleaning of the surfaces may just be necessary. In some cases, ultrasonic techniques can also be used to measure the thickness of a component area where corrosion is present. A small probe should be used where the surface is rough, but in cases of severe corrosion, a measurement may not be possible. Where the transit time for the ultrasonic pulse can be measured, the thickness is calculated by multiplying by the velocity of sound in the material under examination. Some instruments can be calibrated to read thickness directly. Because sound is reflected from the first interior surface encountered, the component being measured must be free from internal flaws. The use of ultrasonic techniques to detect flaws is discussed elsewhere in this chapter.

X-Ray and Gamma-Ray Examination

If X rays or gamma rays are beamed through a component containing defects, such as cavities or cracks, the absorption of radiation at the region of the defects is less than the adjacent sound material. The radiation passing through the component is recorded on a film or a fluorescent screen; the image is darker in the areas of lower density where the defects are located. The image on film provides a permanent record of the defect and also shows the size and shape of the defect in two dimensions. It does not show its position in depth in the component.

Gamma rays having a shorter wavelength will penetrate better than X rays although they are subject to greater diffusion and

consequent lack of contrast. Both types of radiation are well suited to detecting such defects as slag inclusions or porosity in welds. Planar defects, such as cracks, are also detectable but only if oriented approximately parallel to the X-ray beam. Cracks or planar defects perpendicular to the X-ray beam axis will not change the radiation absorption significantly and thus will be undetected. Intermediate orientations will produce varying degrees of defect detectability.

It is estimated that defects that have a depth of two percent of the thickness of a member should be detectable under field conditions (*104*). The accuracy of crack detection is dependent on both the size of the section examined and the location of the crack. For example, a fatigue crack starting at the end of a cover plate on a girder and located directly over the web would not be detected because the X rays would pass through the web section and be absorbed. However, under good conditions cracks of the order of 0.05-in. (1.3-mm) deep and 0.1-in. (2.5-mm) long should be detectable in typical 5/8-in. (15-mm) to 1-in. (25-mm) thick steel sections.

The advantages of X-ray or gamma-ray examination are that they produce a permanent record (when film is used), they have the ability to determine internal defect size and shape (and thus defect nature), and there is widespread acceptability of the validity of the techniques. A particularly attractive feature is that the resulting film is, in fact, a photograph and thus a scaled image of the component examined. No secondary analysis of the data is required. The prime technical disadvantages are the inability to locate the depth of the defect and to locate poorly oriented planar defects. There is also the serious practical limitation that the equipment is relatively large and hazardous to operate such that it may not be possible to use it in all field situations. The results of the survey reported in Appendix A showed that few states use X rays routinely in the field.

Ultrasonic Testing

Ultrasonic testing relies on the fact that discontinuities will produce reflections that are related to their size and shape. The instrumentation usually consists of a piezoelectric transducer or search unit, amplification circuitry, a pick-up device (often part of the search unit), and a cathodic-ray display tube. Waves in the 2 to 6 MHz frequency range have been found to combine good directional properties with adequate penetration (*110*) and are usually employed in field tests.

There are several techniques for using ultrasonic testing to detect flaws in metals. However, many of these techniques are better suited to laboratory work or to specialized applications, such as nuclear pressure vessels or pipeline girth welds, than to highway structures. A typical test arrangement for field use is shown in Figure 8. The ultrasonic pulse is reflected from a flaw

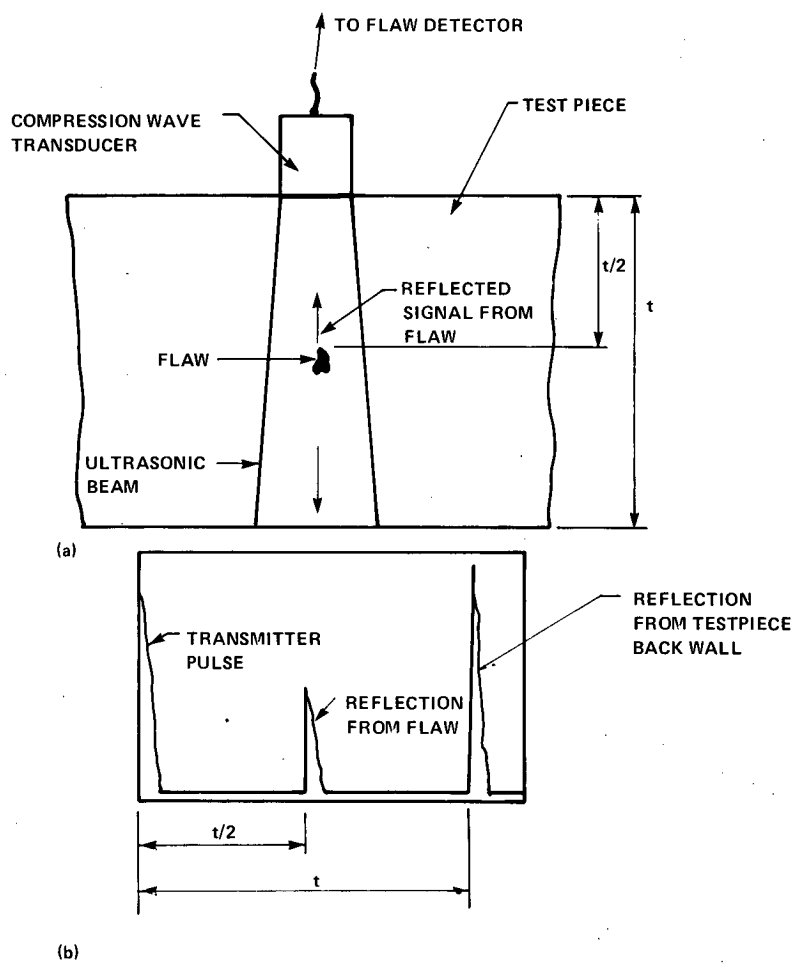

FIGURE 8 Elements of an ultrasonic test: (a) transducer on test piece, (b) CRT display on flaw detector.

and displayed on the screen where the horizontal axis represents the distance from the transducer to the flaw and is calibrated directly in inches (or mm). The vertical axis represents signal amplitude, which is normally measured by comparison with a standard reflector. The differences are recorded in decibels (dB) or as a percentage. The transducer is moved across the surface under investigation and the size of the defects determined. Light oil makes a good coupling medium on steel. The signals appearing on the display screen are recorded photographically or in writing when the operator considers them significant. The number of photographs that would be required is often large and hence this method of recording data is not popular. The test sensitivity is influenced by a large number of testing variables, such as pulse frequency, design of the search head, electronic processing of the return signal, and skill of the operator. Because the velocity of sound in a material and the response of many instruments vary with temperature, frequent calibration is necessary when working under conditions of fluctuating temperature.

Ultrasonic techniques can be used to detect both hidden and surface defects although they are less well suited to surface flaws. The minimum size of cracks that can be detected under ideal field conditions is estimated to be 0.05-in. (1.3-mm) deep and about 0.1-in. (2.5-mm) long (*104*).

The major advantages of ultrasonic testing are portability, sensitivity, and ability to detect the location of cracks or defects with depth. More research has been undertaken to modify ultrasonic testing techniques for improved flaw characterization (e.g., size), including computer enhancement of the pulse-echo data (*110*) so that advances in technology are more likely in this field than in other nondestructive testing techniques. The major disadvantage is that the results are strongly influenced by the skill of the operator. Other negative features are that the signal amplitude is not directly proportional to the size of the flaw and that the sensitivity can sometimes be too high so that too much information, such as grain boundaries and very minor defects not observable by other methods, can cloud the overall picture.

Penetrating Dyes

The dye-penetrant method of inspection is probably the most commonly employed field method of defect detection. Although it is limited to defects that are exposed on the surface of the structure, it is inexpensive, easily applied, and easily interpreted.

The surface of the part of the structure to be examined is first cleaned, either mechanically or with chemicals. A fluid is then placed on the surface under examination, often with an aerosol spray, and allowed to penetrate cracks or surface defects. After a short period of time, the penetrant is wiped off the surface and a second solution, called a developer, is sprayed on. The developer usually dries to a chalky powder and remains unchanged in the regions where no defect exists. In the location of a crack, the penetrant seeps from the crack and stains the developer. The process can be accelerated by striking the part to induce vibration, which will force the dye out of the defects. Bright-colored (often red) penetrants are used to provide a contrast with the white, chalky developer. The defect can then be readily examined and either a photographic or written record made. Modifications of the sytem include penetrants of different viscosity and surface tension to detect different size cracks, wet rather than dry developers, and penetrants that fluoresce under ultraviolet light. Although the penetrants used in conjunction with ultraviolet light examination make smaller defects visible, they are used infrequently in the field.

There is little quantitative information on the accuracy of dye penetrant examinations although it is estimated that, under ideal conditions, the smallest cracks that can be detected are comparable to the capabilities of ultrasonic test equipment.

The principal advantages of penetrating dyes are the ease with which the tests are conducted, the minimal skills required, and the low cost. Tests can be carried out rapidly and in any location that is accessible by the inspector. The principal disadvantages are that only surface defects are detected and it is not possible to assess the depth of the crack. Dye penetrants are also difficult and messy to work with under windy conditions, in which case a careful visual examination with a magnifying glass may be an adequate substitute (*103*).

Magnetic Particles

This method of examination is limited to surface and near-surface defects. In addition, only magnetic materials can be examined. In field applications, the part to be examined is locally magnetized by the use of two current-carrying copper or aluminum rods that are placed on the surface of the part. These rods produce a circular magnetic field about each contact point when current flows between them. Iron powder is sprayed (in suspension) or blown on to the surface and the particles align themselves in the magnetic field. If there are no defects the lines of force are undisturbed but if a defect (such as a crack) is present, the magnetic field is disturbed and a concentration of the iron powder will occur as the powder tries to pile up and bridge discontinuities. This concentration will indicate the presence of the crack or other defect during visual inspection. Under ideal conditions, a surface crack is indicated by a line of fine particles following the outline of the crack, and a subsurface defect by a fuzzy collection of the fine particles on the surface near the defect. To detect a crack, it must be aligned transverse or nearly transverse to the magnetic field. Consequently, the rods are moved about the part of the structure under examination so that defects at any orientation may be detected.

The advantages of the method are its relative portability, the minimal skills required to operate it, and its ability to detect even fine cracks. A major disadvantage is that any paint must first be removed. It also leaves the part examined in a magnetized condition. This is rarely a problem but it may interfere with some subsequent operations such as welding. Although it is possible to demagnetize the affected area, this is time-consuming and adds to the cost.

Eddy Currents

This method is similar to magnetic particle examination except that the defect is detected by a perturbation in an electrical, not magnetic, field in the part under examination. A coil carrying alternating current produces eddy currents in a conductor nearby. The conductor eddy currents, in turn, create impedance in the exciting coil. Alternatively, a separate search coil may be

used. The impedance produced depends on the nature of the conductor and the exciting coil, the magnitude and the frequency of the current, and the presence of discontinuities in the conductor. The equipment is calibrated so that a coil is scanned over the surface of the area to be examined and any defects present produce a characteristic change in impedance, which is read directly from a meter.

The method has received only limited application, primarily because it is only suitable for sections of simple geometry. Complex geometrics change the impedance themselves and thus mask the effect of defects. It does have the advantage that hidden defects can be detected or, with suitable frequency control, examination may be limited to the surface. Defect size can also be estimated. The method is insensitive to many surface conditions (for example, paint) that limit some other methods. Consequently the method does have potential but only the state of Maryland reported using eddy current techniques routinely and there is a need for further development before it has widespread application.

Electromagnetic Induction

Electromagnetic induction techniques have been used in France to examine cables, strands, and wire ropes for corrosion (*6*). The method, which requires a power supply of relatively high frequency, has been applied on the cables of several suspension bridges and has reported to give results approximately 75 percent reliable. Details of the experimental procedures used or of any attempts to refine the method are not known.

Acoustic Emission

The acoustic emission technique involves monitoring the response of a component in the acoustic frequency range to transient elastic waves generated by the rapid release of energy from a localized source in the component. Consequently the principle involved is that the component is subjected to a stress of short duration and the acoustic response is recorded. The main application of the technique in highway structures has been investigations of its potential for detecting crack growth in structural steelwork (*44, 111–113*) and for detecting corrosion in the cables of suspension and cable-stayed bridges (*77, 114*). The technique has been mainly used in materials research and laboratory studies on metals (*10, 115, 116*) but also has application in geological studies (*10, 115, 117*).

When examining cables for corrosion, a satisfactory way of applying stress to the cable was found to be with an air hammer striking a metal plate held between the hammer and the cable to protect the strands (*77*). An alternative approach is to carry out long-term tests by installing several sensors on a cable and measuring the response under traffic loading over a period of several months (*114*). The response is recorded as an acoustic emission count, which is defined as the number of times the amplitude of the response exceeds a preset threshold value.

Applications of acoustic emission to the detection of cracks, but more particularly to the measurement of crack growth, have been encouraging. Transducers and monitoring equipment have been developed that are suitable for use in the field over extended periods. Advances have been made in the ability to eliminate extraneous noise by filters and signal processing techniques (*112, 113*). These developments, combined with more discriminating data-processing techniques to enhance defect identification and source location, have led to significant improvements in the application of acoustic emission technology. Although acoustic emission must still be considered in the development stage, it is not unrealistic that it will become both economic and practical to attach several transducers to a structure and monitor the acoustic responses at a central receiver/processor (*45*). The method appears to be well suited to detecting defects not readily identified by other methods, such as crack growth, corrosion in cables, or cracks in fasteners, hinge pins, or eye bars (*117*).

Combination Methods

Despite the relatively good defect detection capabilities shown by a variety of nondestructive examination techniques, it is often difficult to obtain good performance on real structures under field conditions. For example, sensitivity may be sacrificed to achieve greater coverage, ease of operation, or portability. Some of the potential accuracy of the equipment may be lost because of limited operator skill, operator discomfort, or the surface condition of the structure. To overcome some of these difficulties, a combination system was developed by the Southwest Research Institute (*118*) for detecting fatigue cracks utilizing both an acoustic crack detector (ACD) and a magnetic crack definer (MCD).

The two-instrument system, intended for use by semi-skilled personnel, was designed to first detect (ACD) and then define (MCD) the length of the crack. Each unit consists of a hand-held probe and back pack. The ACD consists of a 2.25 MHz, 70° wedge, ultrasonic probe calibrated to give a digital display of the distance from the probe to the defect. The instrument also incorporates features to measure the effectiveness of the surface coupling and to indicate the presence of a defect through earphones. The unit operates effectively at 3 to 10 ft (1 to 3 m) from the region to be inspected, depending on the surface conditions. The expected sensitivity under those conditions is to be able to identify a crack 3/4-in. (19-mm) or more long.

Once a crack has been detected, the MCD is used to define its length. The MCD consists of an iron core electromagnet operating on a 106-Hz alternating current. Two differential coil pickups oriented selectively with respect to the driving magnet detect disturbances in the magnetic field when a crack is present. The presence of a crack is signalled by a light on the probe and is heard in an earphone. By following the crack with the probe, the length of the crack can be mapped. The unit was designed to determine crack lengths to within 1/4 in. (6 mm) and operates well even on heavily scaled or old painted surfaces.

The unit was evaluated in initial field trials in ten states but the results were mixed (*104*). Large cracks could be detected but the MCD missed a number of cracks and was unsuccessful in defining the tips of others. It was concluded that, in the absence of highly skilled personnel, crack sizes detectable by field crews must be assumed to be at least 1-in. (25-mm) long and about 1/2-in. (13-mm) deep. This was an order of magnitude larger than the minimum sizes detectable under laboratory conditions. Later field trials proved the equipment capable of detecting much smaller cracks such that the ACD and MCD represent a significant advance in the development of instruments suitable for use in the field.

Comparison of Test Methods

A general summary of the relative merits and capabilities of radiation, ultrasonics, magnetic particles, eddy currents, and penetrants is given in Table 7. Electromagnetic induction and acoustic emission have not been added to the table because of the experimental status of these techniques. Table 7 has been condensed from a more comprehensive table published in the early 1970s and reprinted in 1979 (*104*). It permits a rapid assessment of the strengths and weaknesses of each technique and enables the user to select the methods most suitable for a particular application. Approximate costs of the equipment and operator skill requirements are given in reference *93*.

Radiography is widely used for detecting internal voids and defects in welds but has poor capabilities for detecting surface cracks, especially under field conditions. Ultrasonics, although not ideal, does have an advantage over the other methods in that it is capable of detecting the size and position of hidden and surface flaws. Minimum crack sizes detectable by this method are about 0.1-in. (2.5-mm) long but practical detectable sizes in the field are of the order of 1 in. (25 mm). Because ultrasonic techniques are the subject of continuing research, there is the promise of improved detection and characterization capabilities in the future.

Table 7 indicates that the best methods for surface crack detection are magnetic particle and dye penetrant examination. Of these two, magnetic particle inspection is probably the better because of its ability to detect fine cracks and the fact that surface cleanliness is less important.

TABLE 7

CAPABILITY OF NONDESTRUCTIVE EXAMINATION TECHNIQUES FOR DETECTING DEFECTS IN STEEL STRUCTURES IN FIELD USE

Technique		Capability of Defect Detection[a]									
		Minute Surface Cracks	Deeper Surface Cracks	Internal Cracks	Fatigue Cracks	Internal Voids	Porosity and Slag in Welds	Thickness	Stress Corrosion	Blistering	Corrosion Pits
Radiation		N	F[b]	F[b]	P	G	G	F	F	P	G
Magnetic Particle (A.C.)	Wet	G	G	N	G	N	N	N	G	N	N
	Dry	F	G	N	G	N	N	N	F	N	P
Eddy Current		F	G	N	N	N	P	P	N	N	N
Dye Penetrants		F	G	N	G	N	N	N	G	N	F
Ultrasonics[c]		P	G	G	G	G	F	G	F	F	P

[a]G = good; F = fair; P = poor; N = not suitable.

[b]If beam is parallel to cracks.

[c]Capability varies with equipment and operating mode.

LABORATORY PROCEDURES

Many of the tests performed on steel coupons in the field are also used in the laboratory where the ability to work under ideal conditions leads to a substantial improvement in accuracy. The improvements result from the ability to clean and prepare the surface properly and to hold the specimen in any position, and from improved operator comfort. In addition, further improvements can be made by refining field procedures and using more sophisticated techniques. The main disadvantage in the laboratory testing of steel coupons is that the samples are not as easily removed as is the case with concrete and timber structures. In some cases, repairs may be complex and, consequently, sampling should always be carried out under professional supervision. Standard test methods for use in the laboratory are given in Table 8. Test methods listed in Table 6 that are also suitable for laboratory use are not repeated.

TABLE 8

STANDARD ASTM AND AASHTO TEST METHODS FOR STEEL FOR USE IN THE LABORATORY

Designation[a]	Title
A 370 T 244	Methods and Definitions for Mechanical Testing of Steel Products
E 3	Methods of Preparation of Metallographic Specimens
E 8 T 68	Methods of Tension Testing of Metallic Materials
E 10 T 80	Test Method for Brinnell Hardness of Metallic Materials
E 92	Test Method for Vickers Hardness of Metallic Materials
E 103	Method of Rapid Indentation Testing of Metallic Materials
E 110	Test Method for Indentation Hardness of Metallic Materials by Portable Hardness Testers
E 112	Methods for Determining Average Grain Size
E 340	Method for Macroetching Metals and Alloys
E 384	Test Method for Microhardness of Materials
E 407	Methods for Microetching Metals and Alloys
E 807	Practice for Metallographic Laboratory Evaluation
E 883	Practice for Metallographic Photomicrography

[a]ASTM test methods are designated A or E.
AASHTO test methods are designated T.

Metallographic Examination

Metallographic examination consists of a detailed study of metal and may include identification of the material and grade, any defects present, and a description in terms of such properties as chemical composition, mechanical strain, grain size, and heat treatment. A metallographic examination can be especially useful in some uncommon situations, such as a fire on a bridge (*119*). A detailed description of the use of chemical spot tests for the identification of metals and alloys is given in reference *106*. Numerous procedures can be involved, such as the prep-

aration of polished or etched surfaces and the use of optical or scanning electron microscopes. Such an examination is carried out by a specialized professional and, as is the case with the petrographic examination of concrete and the pathological study of timber, a detailed description of the techniques employed are beyond the scope of this report.

Strength Testing

Coupons can be used to measure the strength of steel in a highway structure or to generate the stress-strain relationship for the material. This type of testing is more commonly associated with older structures where the grade of steel is not known and the load-carrying capacity of the structure must be evaluated. In preparing the sample for test, corrosion deposits can be removed by blast cleaning or pickling and the remaining thickness of metal measured accurately. Where necessary, strength testing can be supplemented by additional physical testing, such as impact toughness or fatigue testing.

Indentation and Rebound Methods

Indentation and rebound methods are used to determine hardness. Although hardness measurements are not directly related to the presence of defects or deterioration, they are useful in identifying the grade of a material.

There are several methods and scales for determining the hardness of metals although all depend on measuring the resistance of the metal to a known static or dynamic force. Hardness tests should be made on a smooth surface and at some distance from any edge. When tests are made on a pitted surface, the location of the point of the hardness tester relative to even minor pits can have a significant effect on the hardness reading. Paint, rust, and mill scale should first be removed. Several measurements should be made with various locations on the surface so that spurious results can be eliminated.

The methods of hardness testing in most common use (*120*) are listed in summary form below. The applicable test methods are included in Table 8.

The Brinell test measures the size of the impression created by a hardened steel ball under load. A 10-mm diameter ball and load of 3000 kg are normally used although a formula is provided for changing both the size of the ball and the load.

The Rockwell method uses a steel ball or a diamond cone as the indentor. A preload of 10 kg is applied followed by either a 100-kg load on the ball (Scale B) or 150 kg on the cone (Scale C). The hardness of the material is then related to the difference in the two depths of indentation.

The Vickers method uses a diamond square-based pyramid as the indentor to reduce deformation errors and to increase the range of materials to which the method is applicable.

The Knoop Pyramidal indentor makes a diamond or rhomboidal impression with the diagonals having a ratio of approximately 7:1. The test is normally done under a force of less than 1 kg and is suitable for thin specimens.

The Monotion Test involves the measurement of the force required to create an impression 0.0006-in. deep with a diamond hemisphere 0.75 mm in diameter. This test is suitable for use on thin metals and hard steels.

The Shore scleroscope test is performed by measuring the height of the rebound of a small diamond-pointed hammer falling within a glass tube from a specified height. A smooth surface is necessary to achieve precise results.

The Herbert test is based on the measurement of the amplitude or period from a given position of a small steel or diamond ball on a level surface.

Radiography

The results of X-ray and gamma radiography are generally much more satisfactory in the laboratory than in the field because the working environment is safer and there are no problems handling the equipment. In addition, the sample can be studied at any orientation with respect to the axis of the radiation beam, which is not often possible in the field. Under these controlled conditions, radiography should reveal defects between 0.5 and 1.0 percent of the thickness of the material examined.

Ultrasonics

There are a number of variations on the moving probe technique of ultrasonic testing most commonly used in the field. These include varying the angle of incidence of the beam with respect to the surface of the material, using multiple transducers, and using sophisticated equipment that is not sufficiently rugged for field use. The amplitude assessment technique uses the single parameter of signal amplitude to indicate the severity of defects. This technique is widely used for the acceptance and rejection of welds at the shop fabrication stage. A newer development is the time-of-flight technique, which is based on the principle that diffraction effects at the ends of flaws cause secondary signals to be radiated. Their signals can be detected by a second transducer so that the time of flight from the transmitter to the flaw tip to the receiver can be used to position the flaw tip accurately. This particular technique shows promise for the reliable detection and measurement of flaws (*110*). A comparison review of crack-depth measurement using a variety of ultrasonic techniques is given in reference *121*.

Penetrating Dyes

The use of penetrating dyes in the laboratory leads to better results and the ability to identify smaller defects than is possible in the field. These improvements are possible because the surface can be better prepared and more sophisticated techniques with low viscosity dyes and ultraviolet light detection can be utilized.

Magnetic Particles

Magnetic particle examination in the laboratory is conducted by placing the sample in an apparatus that permits the use of a more powerful magnetic field than is possible with portable equipment. To ensure that defects are aligned perpendicular to the magnetic field, the sample is sequentially magnetized by placing it in a large circular coil to produce a longitudinal magnetic field and passing current through the part to produce

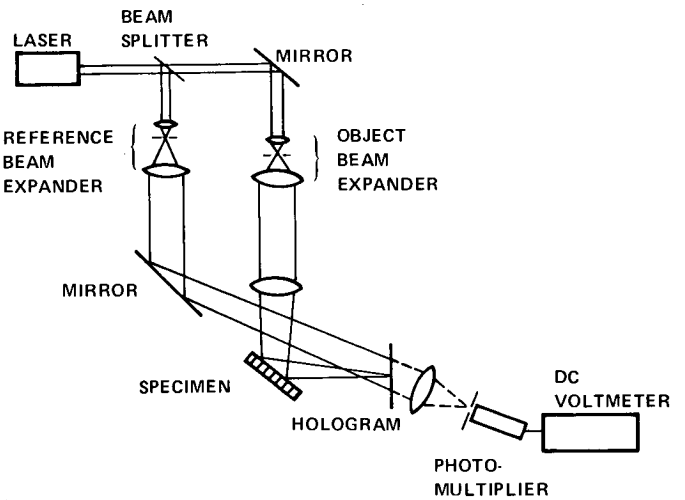

FIGURE 9 Arrangement of apparatus for constructing holograms.

a circular magnetic field. If this apparatus is used with a wet system of applying iron powder, very fine surface cracks can be detected.

Holography

Optical holography is a two-stage process that permits the reconstruction of three-dimensional images (*122*). A typical arrangement of the apparatus for constructing a hologram is shown in Figure 9. It consists of illuminating the object to be examined with a coherent optical radiation source (laser) causing the scattered beam from the object to interfere with a reference beam obtained from the same source. The resulting interference pattern is recorded on photographic emulsion. The second or reconstruction phase is to illuminate the negative with a suitable laser source to produce a three-dimensional image of the original object. A modification of the method is to produce multiple images when the object is subjected to small applied loads. Illumination of the negative then reconstructs the image with interference fringes corresponding to the displacement of the object surface in successive exposures. The method is extremely sensitive to small displacements because the interferograms are caused by changes in the optical path length of the order of one wavelength of light (*123*).

Holography is a sophisticated laboratory technique that has been used for the examination of defects, strains, and vibration modes of such components as brake disks, tires, and gas turbine blades. The method could be used to detect cracks and hidden anomalies in coupons cut from highway structures but there are no reports of such applications and the technique is better suited to research and quality control of mechanical components.

CHAPTER FIVE

TIMBER COMPONENTS

CHARACTERISTICS OF TIMBER

Timber components in highway structures are characterized by variability because of the use of different species, varying grades, and the presence of defects and deterioration.

The main constituents of timber are cellulose, which is a carbohydrate that forms the framework of the wood cells, and lignin, which is an irregular polymer that acts as a thin cementing layer between the cells. The fact that timber is a cellular, organic material also means that it is porous, subject to decay, and very vulnerable to damage by fire. It is also elastic and low in themal and electrical conductivity but is subject to relatively large volume changes in response to changes in humidity. Under certain conditions, and when properly treated with preservative and protected, timber is quite durable.

Timber is strongly anisotropic, having vastly differing properties parallel and perpendicular to the grain. In contrast to concrete and steel, the single indicator of the overall quality of timber is its flexural strength. The allowable compressive strength is approximately 75 percent of the flexural strength when loaded parallel to the grain but only 20 percent when loaded perpendicular to the grain. It is relatively weak in shear, having a shear strength approximately 10 percent of the flexural strength. It should be noted that, in contrast to concrete and steel where allowable stresses have tended to increase over the years, allowable stresses in some species have decreased. This results partly from the fact that the best timber has already been harvested and partly from a better understanding of structural mechanics. There has also been a decrease in the size of members of the same nominal size. An important attribute of timber is that it is able to withstand significant increases in load momentarily (*15*) and consequently neither impact nor fatigue are serious problems in timber structures.

COMMON DEFECTS AND TYPES OF DETERIORATION

The most common defects and types of deterioration in timber structures are described below. Illustrations of the defects are given in references *4* and *7*.

Fungi Decay

Fungi require moisture, oxygen, and mild temperatures to flourish (*124*). Mild fungus attack appears as a stain or discoloration. It is sometimes difficult to distinguish between fungi that cause decay and those that simply stain the timber, although, as a general rule, mold and fungus stains are confined to sapwood. Stains and molds should not be considered stages of decay as the causal fungi ordinarily do not attack the wood appreciably.

In contrast, decay-producing fungi, especially in the more advanced stages, cause the wood to darken and show definite signs of deterioration. The surface may become soft and spongy, stringy, or crumbly depending on the type of fungus. In some cases, fruiting bodies of fungi may be readily visible. Decay fungi may attack heartwood or sapwood. The fungi, in the form of microscopic thread-like strands, permeate the wood and use part of it as food. Some fungi live largely on cellulose and cause "brown rot," so called because the wood takes on a brown color. The wood also tends to crack across the grain and to shrink and collapse. Other fungi feed on both lignin and cellulose and cause "white rot" in which the wood may lose color and appear lighter colored than normal. White rot does not result in cracks across the grain and, until severely degraded, the wood retains its outward dimensions and does not shrink or collapse.

For the most part, decay is relatively slow at temperatures below 10° C (50° F) and above 32° C (90° F) and essentially ceases when the temperature drops below 2° C (35° F) or rises above 38° C (100° F) (*124*). Serious decay occurs only when the moisture content of the wood is above the fiber saturation point (about 30 percent). However, if the wood is saturated, there may be insufficient oxygen to support the development of decay fungi. Decay is rare at moisture contents less than about 20 percent. Decay is most likely to occur in highway structures where moisture is retained as, for example, at connections, splices, support points, or around bolt holes and spikes.

Vermin

There are several kinds of vermin that will tunnel in and hollow out the insides of timber members for food and shelter. Vermin attack is most prevalent where the climate is hot and in timber in salt water. The most common vermin are termites, powder-post beetles, carpenter ants, and marine borers.

Although about 56 species of termite are known in the United States, they can be conveniently divided into two groups from the standpoint of the methods of attack in wood: the ground-inhabiting or subterranean termites and the wood-inhabiting or nonsubterranean termites. Subterranean termites are found in most parts of the United States but are much more prevalent in the southern states. These termites develop their colonies in the ground and the worker members attack adjacent structures for food. The termites make galleries that generally follow the grain of the wood, leaving a shell of sound wood to conceal their activities. Evidence of termite infestation are white mud shelter tubes or runways extending up from the earth. Excessive sagging or crushing of members may indicate advanced stages of termite attack.

Nonsubterranean termites have been found only in a narrow strip extending around the southern edge of the United States from California to Virginia with the worst damage being found

in southern California and southern Florida. Unlike the subterranean termites, which require a constant source of moisture in their colonies, the nonsubterranean termites can live in dry wood without moisture or contact with the ground. Although the total amount of destruction they cause in the United States is much less than that caused by subterranean termites, they are a definite menace in the regions where they do occur.

Powder-post beetles attack both hardwoods and softwoods. Eggs are laid in pores of the wood and the larvae burrow through the wood, making tunnels about 2-mm (0.08-in.) diameter, which they leave packed with a fine powder. Evidence of infestation is found in the form of holes left in the surface of the wood by the winged adults as they emerge and by the powdery dust, which may be visible around the holes. Serious attack by powder-post beetles can result in the interior of a member being completely excavated.

Carpenter ants are black or brown. They tend to use wood for shelter rather than for food, usually preferring wood that is naturally soft or made soft by decay (*124*). Evidence of carpenter ant activity is the presence of the ants themselves on the structure or the accumulation of sawdust around a member.

Damage by marine-boring organisms is widespread in salt or brackish waters. Slight attack is sometimes found in rivers upstream from brackish conditions. Along the Pacific, Gulf, and South Atlantic coasts, attack is rapid and, although slower, marine borer attack is still a serious concern along the north Atlantic coast. The action of marine borers is usually most severe in the part of members between high and low water although damage can, on occasion, extend to the mud line.

Mollusk borers or shipworms are the most serious enemies of marine timber installations. The most common shipworm is the teredo, which enters the timber in an early stage of life and remains there throughout its life. The teredo maintains a small opening in the surface of the wood to obtain nourishment from the sea water. Teredos reach a length of 15 in. (380 mm) and a diameter of 3/8 in. (10 mm). Some species of shipworm grow even larger. A serious infestation of shipworms can result in complete disintegration of a timber pile.

Crustacean borers bore into the surface of the wood to a shallow depth. Wave action or floating debris then breaks down the thin shell of timber outside the borers' burrows causing them to burrow deeper. Continuous burrowing results in progressive deterioration of timber piles and a characteristic hour-glass appearance between high and low water because of the reduction in cross section.

Wood-boring mollusks or pholads resemble clams and, like shipworms, enter the wood when very small, leaving a small entrance hole. They generally do not exceed 2.3 in. (60 mm) in length and 1 in. (25 mm) in diameter but are capable of doing considerable damage. In the United States, pholad activity appears to be confined to the Gulf of Mexico (*124*).

Weathering

Weathering is caused by repeated dimensional change in the wood, usually as the result of wetting and drying. Given the poor thermal conductivity of wood, timber structures tend to contract in hot weather (when shrinkage caused by drying is greater than thermal expansion) and expand in cold wet weather (when the swelling effect prevails over that of cooling) (*125*). Such movements can cause not only loosening of bolts and joints but induced stresses in the timber. In the early stages of deterioration the surfaces of the wood are rough and corrugated as a result of the swelling of the fibers. At more advanced stages, large cracks or checks may extend deeply or completely through the wood. Warping of the member may occur and, in extreme cases, the wood may crumble.

Chemical Attack

There are three different mechanisms by which chemicals can cause deterioration of wood. Wetting may cause swelling and a resultant weakening of the fibers, acids can cause hydrolysis of the cellulose, and alkalis can cause delignification. The most common source of aggressive chemicals are spillage, industrial pollution, and animal wastes. Unlike concrete and steel, timber is not affected by deicing salts, which act as a preservative. The physical manifestation of chemical attack is similar to decay.

Fire

Timber is particularly vulnerable to fire, although the presence of charred members is easily recognized.

Abrasion or Mechanical Wear

Wear caused by abrasion is most common on bridge decks and is readily recognized by the gradual loss of section at the points of wear, notably the wheel tracks. Abrasion is most serious when combined with decay, which softens and weakens the wood.

Collisions and Overloadings

Damage can take the form of shattered, splintered, or deformed timber, large longitudinal cracks, or sagging and buckled members. Even relatively minor damage may break through the protective treatment and expose untreated wood to subsequent decay.

FIELD PROCEDURES

In general, the state of the art in test methods for detecting defects and deterioration in timber structures is not as advanced as for concrete and steel structures. There are fewer test procedures available and no standard test methods specifically written for testing timber components in service. However, many of the procedures contained in ASTM D 245, "Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber," can be adapted for use on timber highway bridges.

Visual Inspection

The visual inspection includes the identification and recording of those parts of the structure exhibiting the defects and types of deterioration described above.

It is not usually possible to detect the early stages of decay from the visual appearance of the wood. At more advanced stages, the wood may lose its characteristic luster and darken in color, although this is not always the case. However, a number of conditions can be identified that are conducive to decay and parts of the structure exhibiting these conditions should be inspected carefully. Dampness, rust stains, or the growth of moss or other vegetation (especially in cracks) is evidence of potentially hazardous wetting. Bearing areas where moisture may be trapped or areas where dirt may accumulate and hold moisture and initiate decay need careful examination. Nails, drift pins, and other fasteners also represent areas where water may have entered the structure causing wet areas within members and the possibility of internal decay.

Connections are important because a reduction in the load-carrying capacity of a timber structure often begins with deterioration of the connections. Particular attention should be paid to evidence of loose fit or softening at bolt holes and splices. Fasteners and hardware should be checked for signs of corrosion. Excessive vibration or noise can sometimes be an indication of deficient connections or loose members. Other examples of loss of integrity at connections are gaps that can occur at the node points of trusses or independent movement of planks in nailed, laminated decks.

Bituminous surfacings can complicate the inspection of timber decks. As in the case of concrete decks, the bituminous surfacing should be removed in selected areas and more attention should be paid to the condition of the deck soffit. Water stains on the soffit are often evidence in laminated decks of the joints opening between the laminations.

Careful visual inspection will reveal the presence and size of defects, such as knots and checking, and the slope of the grain. Members should also be observed for signs of overstressing, excessive deflection, or misalignment. These observations are particularly important where an evaluation of the load-carrying capacity of an individual member or of the entire structure is to be undertaken. In such cases it will normally be necessary to identify the species and the stress grade of the timber. This can sometimes be done from observation of the surface characteristics of the timber (*126*). Where uncertainty exists, conservative assumptions must be made or a sample must be removed for identification or strength testing.

It is usually possible to determine whether a preservative has been used, but it is often difficult to identify the type of treatment involved. This is important because pressure-treated timber is much more resistant to decay than timber dipped in a bath of preservative, often creosote. Timber that has been pressure treated can sometimes be distinguished by small incisor marks in the surface. These cuts facilitate penetration of the preservative into the wood. However, the lack of marks does not mean that the timber was not pressure treated. Another reason for identifying preservatives is that some treatments reduce the strength of the timber (*4*), particularly oil treatments, which soften the wood surface (*127*), and salt treatments, such as ammoniacal or chromated copper arsenate, which cause embrittlement (*128*). Wood can be expected to lose 5 percent of its strength and undergo about 15 percent increase in brittleness for every 1.0 lb/ft^3 (16 kg/m^3) of salt retained (*1*).

Glued-laminated structures require the additional inspection of the condition of the glue lines. Any longitudinal separations must be examined to determine if the separation is in the glue line or in the wood. Failure in the wood should be treated as equivalent to a seasoning check in sawn timber (*129*). Glue failure, indicated by the presence of glue on one or both surfaces of the opening, could be an indication of serious deficiencies and must be thoroughly investigated. Feeler gauges can be used to probe glue-line gaps to determine their minimum depths. However, visual examination would normally be supplemented by taking plugs through the glue line for laboratory examination or shear testing (*129*).

Depth of Penetration, Drilling, and Coring

Any probe, such as a knife, ice pick, nail, or brace and bit, can be used to test for internal decay or vermin infestation (*4, 129, 130*). The ease with which a member can be penetrated is then a measure of its soundness. Only a qualitative assessment is obtained because the pressure on the instrument is neither controlled nor measured. Although the procedure is rather crude, it is rapid and an overall assessment of the condition of a structure can be obtained quickly. The use of a probe is much more satisfactory than attempting to identify a hollow member by sounding because the load on the member affects the response and may lead to erroneous conclusions (*127*).

A review of nondestructive test methods by the Forest Products Laboratory (*128*) identified work in Sweden to develop an instrument called a "Pilodyn" for estimating strength loss in timber. The instrument operates on the same principle as the Windsor Probe and measures the depth of penetration of a pointed pin shot into the wood. The penetration is used for estimating the degree of decay (soft rot in the Swedish studies), which in turn is correlated with loss of strength.

An increment borer, which consists of a sharpened hollow tube, usually about 1/4-in. (6-mm) internal diameter, can also be used to penetrate the wood. The borer is superior to a nail or ice pick because it gives a more accurate record of the depth of decay or infestation. It also allows samples to be removed from the interior of the member for detailed examination for testing for such items as moisture content and preservative penetration, or to be cultured for positive evidence of decay fungi.

A hand instrument has been developed (*131*) that allows an estimate of the compressive strength of the timber removed by a borer. It was developed primarily to select timber of high fiber strength in the woods. The device resembles a pair of pliers with a spring system to measure the squeezing force. The core is placed between the jaws of the instrument so that the grain is parallel to the jaws and a given force applied to the handles. The amount of compression is measured by a dial gauge. The compression reading, within the elastic limit of the wood, is then directly correlated to the strength of the member.

Electrical Methods

The main application of electrical methods is to measure the moisture content of timber. The moisture content of wood is a key indicator of the potential durability of wood because moisture is not only associated with rot and fungal attack but renders timber more vulnerable to penetration by vermin. Moisture content is also important because of its effect on the strength of timber. An increase in moisture content of one percent will

result in approximately five percent decrease in strength (*15*). A number of cases of poor durability performance of timber structures can be traced to improper drying of the members before construction. There are several electrical techniques available for measuring moisture content (*132*).

Resistance meters are based on a direct current measurement of electrical resistance between point or blade electrodes pushed into the timber. The resistance is related to the moisture content, which is displayed on a calibrated scale. The results are affected by the species of timber and correction factors must be applied. Resistance moisture meters are light, compact, and inexpensive but the major disadvantage is that they measure the moisture content of the surface layers unless special deep probes are used. Readings over 30 percent moisture content are not reliable and contamination by some chemicals, such as salt, affects the readings.

Capacitance meters are based on an alternating current measure of the dielectric constant of wood, which is proportional to its moisture content. The results are a function of the relative density of the wood and correction factors must be applied. The meters measure primarily surface moisture content, and, on lumber thicker than 2 in. (50 mm), do not respond to internal moisture adequately (*133*). Capacitance meters have a wider range (0 to at least 35 percent moisture content) than resistance meters and are less affected by the presence of chemicals.

Radio frequency power-loss meters operate in the frequency range 0 to 25 MHz and are based on an alternating current measurement of the impedance (combined effect of resistance) and capacitance of timber. They need to be calibrated for wood species and density. The meters use plate-type electrodes and the field penetrates about 3/4 in. (20 mm) but the surface layers have the predominant effect. The cost of the meters is similar to that of capacity-type meters, being higher than that of simple resistance types. Other more sophisticated meters are available (*132*) but are better suited to the laboratory than field use.

Electrical resistance measurements are also the basis of an instrument designed to detect internal rot. The device consists of a resistance probe, which is inserted to various depths in a hole 3/32 in. (2.4 mm) in diameter. A marked change in electrical resistance is an indication of decay. Although the device effectively detects rot, it is susceptible to false indications of decay in apparently sound wood (*129*).

Sonic Methods

Sonic methods have been applied in the field to the detection of decay in utility poles (*129, 134*) but no record has been found of their application to timber highway structures. Because of their accessibility and the simplicity of the support conditions, utility poles are easier to evaluate. A number of test methods are possible but all involve either the measurement of the velocity of a sound wave through the timber or determination of the natural frequency of vibration. A low reading in either case was found to be associated with the presence of internal decay.

Ultrasonic Techniques

The same ultrasonic pulse-velocity equipment and techniques described in Chapter 3 for application to concrete members can also be used for the in-situ testing of timber structures (*135*). The equipment has a number of important attributes:

a. The shape and size of the member does not present serious restrictions.

b. The method does not induce additional stresses in a member already under load.

c. Measurements can be checked and rechecked without changing the condition of the timber.

d. Changes over the life of a structure can be monitored.

e. Measurements can be made rapidly on any accessible plane surface so that a detailed survey of all parts of an extensive structure is often feasible.

f. The test method is entirely nondestructive.

Pulse-velocity measurements relate to the elastic properties of the wood and are therefore sensitive to the direction of the grain. However, pulse-velocity measurements have been found to follow similar trends to strength changes caused by fluctuations in density and local defects (*135*). Consequently, the strength and stiffness properties of the timber can be assessed. The ultrasonic method can also be used to identify internal decay and hollow areas, as well as internal knots, checks, and shakes (*136*). Because a discontinuity, such as a crack or a hollow area caused by decay, reflects part of the sound wave and changes the velocity of the transmitted wave, the technique is most sensitive to detecting defects that are oriented perpendicularly to the pulse. For this reason, the direct transmission mode with transducers on opposite faces of the member is generally the most useful configuration. However, in some situations, it may be necessary to investigate other relative positions of the transducers in order to produce a maximum response. To simplify interpretation of the results it is common practice to compare the pulse velocity from a suspected area of deterioration with that from an area known to be sound (measured using the same transducer configuration), thereby eliminating the need to measure the density of the timber. In all cases, a good contact between the transducer and the surface of the timber is essential (*137*). A light grease or glycerol are suitable for the coupling medium. Bentonite paste has also been found satisfactory (*138*).

It is desirable that the frequency of the ultrasonic waves be as high as possible to preserve the well-defined onset to the pulse for accurate measurement of the pulse velocity. However, an upper limit to the frequency that can be used is imposed by the attentuation caused by the voids in the internal structure of the wood. The precise frequency at which attenuation becomes serious depends on the species, density, grain direction, and internal structure. In practice, transducers having a frequency range of 50–250 kHz have been used.

The application of ultrasonic measurement techniques to timber is less extensive than to concrete so that some uncertainty still remains in interpreting the results. In an investigation of decay in timber piling (*52*) it was found that the sound, dry timber had a pulse velocity of the order of 6000 to 7000 ft/s (1.8 to 2.1 km/s) but as low as 5500 ft/s (1.7 km/s) when saturated. The velocity of decayed wood when dry could be as low as 4000 ft/s (1.2 km/s) or as high as 5200 ft/s (1.6 km/s) when saturated. As a result, more meaningful results could be obtained in the dry season. It must be remembered that the test does no more than measure the speed at which

sound waves travel through the material in a particular mode and direction. All other information on the strength and condition of the material is then inferred. However, significant progress has been made in using ultrasonic techniques to identify deterioration in timber structures (*135*) including the application to pile supports underwater (*130, 139*) and in locating internal defects at the sawmill (*140*). The use of the method can be expected to become more widespread as experience and confidence increase.

Comparison of Test Methods

A summary of the capabilities of each of the test methods for detecting defects and deterioration is given in Table 9. Approximate equipment costs are given in reference *93*. In contrast with Tables 3 and 7 for concrete and steel structures respectively, the investigative techniques and types of defect are relatively few. A careful visual survey supplemented by a depth of penetration device or increment borer is sufficient to identify and estimate the severity of most types of deterioration. Ultrasonic techniques are a useful supplement, especially where an indication of the strength of the timber is also required. Specific identification of decay fungi, vermin, chemical attack, and preservative treatment or the actual measurement of strength is best done by removing samples for laboratory analysis.

LABORATORY PROCEDURES

As with concrete and steel, the removal of specimens cut from timber structures to the laboratory allows for a much more thorough identification of the material and testing for defects and deterioration than is possible in the field. However, the potential benefits from a laboratory test program must be carefully examined before embarking on such a course of action. Not only must the material and labor cost for removing the timber and making good the structure be considered, but sufficient samples must be taken to be representative of the portion of the structure under examination.

The standard test methods in use are listed in Table 10.

TABLE 9

CAPABILITY OF INVESTIGATIVE TECHNIQUES FOR DETECTING DEFECTS IN TIMBER STRUCTURES IN FIELD USE

	Capability of Defect Detection[a]				
Technique	Surface Decay and Rot	Internal Decay and Voids	Weathering	Chemical Attack	Abrasion and Wear
Visual	G	P	G	F	G
Depth of Penetration	G	G	F	F	N
Moisture Content	F	F	N	N	N
Ultrasonics	N	G	G	N	N

[a]G = Good; F = Fair; P = Poor; N = Not suitable

TABLE 10

STANDARD TEST METHODS FOR TIMBER FOR USE IN THE LABORATORY

Designation	Title
D 143	Method of Testing Small Clear Specimens of Timber
D 198	Method for Static Tests of Timbers in Structural Sizes
D 1860	Test Method for Moisture and Creosote-Type Preservative in Wood[a]
D 2016	Test Methods for Moisture Content of Wood
D 2017	Method for Accelerated Laboratory Test of Natural Decay Resistance of Woods
D 2085	Test Method for Chloride for Calculating Pentachloro-phenol in Solutions or Wood (Lime Ignition Method)
D 2395	Test Methods for Specific Gravity of Wood and Wood-Base Materials
D 2915	Method for Evaluating Allowable Properties for Grades of Structural Lumber
D 3345	Method for Laboratory Evaluation of Wood and Other Cellulosic Materials for Resistance to Termites

[a]Substantially the same as AWPA-A6.

Pathological Examination

The examination under the microscope of prepared surfaces along and across the grain enables a detailed description of the wood to be prepared. The species of timber, and often the stress grade, can be defined. It also permits examination for evidence of overstress, the extent of deterioration to be measured and documented, and the type of fungus or vermin to be identified. Such work is normally carried out by a qualified professional.

Following a visual examination, physical testing can be performed to quantify the structural properties of the timber.

Density

The density of timber is an important property because it also provides an assessment of the flexural strength. At present there is no more accurate method of measuring density than weighing a piece of timber, measuring its volume, determining its moisture content, and calculating its density. A very convenient method is to oven dry the small cores that are removed by an increment borer. Several cores are required to give sufficient weight for reasonable accuracy. At moisture contents well above the fiber saturation point, the results tend to be low because the timber is compressed during the boring process. At moisture contents less than about 18 percent, good agreement with full cross-sectional slices is generally obtained (*132*).

Other methods of measuring density rely on penetrating radiations. As with any material, when a beam of radiation is passed through wood it is gradually absorbed, the degree of attenuation depending on the thickness and density of the material. The amount of penetration is a function of the wavelength of the radiation, the shorter wavelengths penetrating most easily. However, a complicating factor with wood is that it contains water, which also absorbs the radiation, and this must be identified separately and allowed for in density calculations.

Beta radiation penetrates only about 3/8 in. (10 mm) and, although it has been used in the quality control of thin, sheet materials, it is not very useful for field samples. Gamma radiation can penetrate most structural members and can be used to find the density of timber, including the absorbed water. The moisture content must then be determined by other means. Neutron absorption, which offers the potential to be combined with gamma radiation in a single instrument, is a promising technique but the cost of such an apparatus has prevented its development. Such an instrument would allow for the rapid measurement of density and be totally nondestructive. In addition to cost and safety restrictions, the major drawback to radiation techniques is that they cannot be used on irregularly shaped pieces of material.

Moisture Content

In addition to the meters described for use in the field, the moisture content of wood can be determined in the laboratory by oven drying cut samples or by radiation techniques. Where practical, the oven-drying method gives the most accurate and economical result.

The microwave absorption meter, as the name implies, measures the attenuation of microwaves (wavelength about 1 in. or 25 mm) passed through the timber. The moisture is shown on a calibrated dial although a correction for the density of the sample is required. The meter will read up to 50 to 60 percent moisture on thin (less than 1 in. or 25 mm) samples but is most reliable between 30 and 60 percent. The result corresponds to the average rather than the surface moisture content. The equipment is rather bulky and expensive and therefore not in general use.

Neutron-scatter equipment depends on the attenuation of a beam of neutrons by the hydrogen contained in water molecules. The equipment is expensive and is limited to experimental rather than production use.

Strength Testing

Where the stress grade of the timber in a structure is unknown or where an accurate assessment of strength is required for the calculation of load-carrying capacity, a direct measurement of the flexural strength of a beam sample may be possible, although this would normally be done for long or otherwise important structures. When combined with a visual stress-grading system to account for defects (knots, checks, slope of grain), the strength of other members in the structure can then be estimated. Alternatively, members may be removed from the structure and the strength estimated using nondestructive test procedures. The most common method is a simple beam test in which the deflection of the beam is measured for given increments of applied load. The ability to predict the strength of the timber is then predicated on the fact that there is a significant statistical relationship between the ultimate flexural strength of timber and its modulus of elasticity (*141–144*). It has also been found (*145*) that the modulus of elasticity gives a better index of bending strength than does a measure of knot size or knot-area ratio, the factors on which visual stress grading are principally based. This same principle is used in the mechanical grading of timber for use in new construction. A number of machines that either impose a small load on the timber and measure the resulting deflection or measure the load required to produce a specified deflection have been developed for grading purposes (*132, 142, 143, 145, 146*).

A measure of the mechanical properties of timber can also be obtained from the vibrational characteristics of the material (*141, 147*). Subsonic vibration frequencies (less than about 50 Hz) can only be used on small specimens. Sonic frequencies (about 50 to 16,000 Hz) have been used in the development of a machine in which the timber is vibrated in the transverse mode and the period of vibration measured (*148*). The modulus of elasticity is then calculated from the relationship $E = WK/T^2$ (Eq. 4). A refinement of the method in which the rate of decay of free transverse vibrations is measured has been investigated. It is claimed (*147*) that there is a strong correlation between the logarithmic decrement of the vibration and the modulus of rupture of the timber although the relationship has not been universally accepted.

Stress wave propagation is similar to using transverse vibration methods except that ultrasonic frequencies are used (above about 16,000 Hz) and the direction of excitation is longitudinal. In this case, the modulus of elasticity is calculated from the relationship expressed by transposing Eq. 2:

$$E = C^2 D \tag{6}$$

which is valid for a prismatic bar

where

E = dynamic modulus of elasticity,
C = velocity of propagation, and
D = density of the material.

A more complete mathematical treatment of the different modes of vibration and the effect of end conditions is given in reference *149*. The transverse and longitudinal vibrations yield very similar values of the dynamic modulus of elasticity, which correlate very well with the average static modulus of elasticity along the length of a member (*144*). Vibration methods are not very effective at detecting weak spots or defects in members. This is important in highway structures where most members are individually loaded (as opposed to a load-sharing structural system) because a defect could be the determining factor in the strength of a member.

Radiography

X rays can be useful for investigating the internal condition of timber. Because timber has a low X-ray absorptive capacity, best results are obtained using low-energy X rays (generally in

the range 10–65 kV). However, low-energy X rays generate more secondary radiation than high-energy X rays and this reduces image contrast. A laboratory study to determine optimum exposure techniques and the use of filters to absorb secondary radiation is described in reference *150*. This study also included the examination of 22 utility poles in the field. Although radiography was found to be feasible and the results were encouraging, it is rarely used in practice. This appears to be a reflection of the high cost of the equipment, the safety regulations that govern its use, and the expertise required to interpret the radiographs.

Computed tomography is being studied for its potential applications to timber and wood products. It is being used in a pilot study to identify defects in logs at a sawmill in British Columbia so that the logs can be cut for the optimum end use. The U.S. Navy is investigating the feasibility of the technique for underwater use (*1*). The equipment could also be used to identify defects in samples taken into the laboratory and ultimately, may be developed into equipment suitable for field use, although the cost would limit its application to special cases.

The application of gamma radiography to wood has also been investigated but with limited success (*130*). The inherent variability of wood and the presence of many naturally occurring discontinuities makes it much more difficult to apply radiation techniques successfully to wood than to homogeneous and isotropic materials.

CHAPTER SIX

FULL-SCALE TESTING OF BRIDGES

INTRODUCTION

Techniques for monitoring the overall condition of a bridge present possibilities of making an assessment of its condition, or changes in its condition, and of detecting faults. It is often very difficult to analyze the effects of defects or deterioration on the overall performance of a bridge or on the stresses in individual components. Although load testing is the most common example of a full-scale test, measurement of change in geometry or the response of a structure to vibration can also be useful techniques in some situations.

LOAD TESTING

In the early days of bridge testing, most tests were proof tests to ensure that a bridge could carry the assumed load level. More recently, bridge testing has been used in studies of load distribution, in the measurement of stress in members under traffic loading, in research studies of ultimate capacity, and in ascertaining dynamic response (*120*).

Bridge testing is both an art and a science. In its simplest form, load testing involves measuring the response of the bridge to a known applied load. Considerable experience is required to know where to locate gauges and to determine load increments and the maximum load to be applied to prevent damage to the bridge. Strain or transducer gauges are commonly applied to the elements under investigation in the areas of maximum stress. More modern developments have been laser beam deflection recording equipment and reusable strain gauges. The load is applied, usually by vehicles although occasionally by dead load or through cables, to induce maximum effects. Where a measurement of stress at a given location under known load is all that is required, data processing may not be a major consideration. In other cases, the amount of data recorded is often extensive such that automatic data recording and analysis is highly desirable. It is also preferable that this be done on site as the test progresses so that deviations from anticipated behavior are known and any necessary changes in procedure or equipment can be made. Because concrete and timber properties are required to define stress values (from strain measurements), samples must be taken and tested in accordance with the procedures defined in Chapters 3 and 6. In steel and wrought-iron bridges, the modulus of elasticity can be estimated sufficiently reliably for most purposes.

Load tests are not cheap. On larger structures they require considerable planning, involve many people, and demand the use of sophisticated equipment (*151*). Testing in remote locations can present additional difficulties (*152*). However, load testing can often be justified where the effect of defects or deterioration on load capacity cannot be determined by analysis. There are a number of cases where the load capacity of deteriorated bridges, especially short-span concrete bridges, has been found to be much higher than predicted such that posting or replacement was not required (*120, 151, 153, 154*). Alternatively, load testing can sometimes be used to determine why certain defects (although not environmentally induced deterioration) have developed. However, a decision to carry out full-scale load testing should not be undertaken lightly and there are several publications that contain detailed information on procedures, instrumentation, and case studies (*120, 151, 153, 155–160*).

CHANGES IN GEOMETRY

Long-term monitoring of bridge geometry can also be useful in determining the cause of defects such as cracks induced by

unanticipated movement of foundations or loss of prestress. Structural faults are detected by noting movements with respect to a plane of reference. Such monitoring may range in complexity from simple scratch gauges at joint locations to regular surveys of a structure using precise survey equipment, laser techniques, or photogrammetry techniques. This type of long-term monitoring is mainly applicable to medium and long-span bridges where geometric changes are likely to be large enough to be measured with sufficient accuracy. In short-span concrete and composite structures, the geometric changes resulting from even substantial flaws or loss of material are likely to be so small as to be difficult to distinguish from thermal effects (*3*).

RESPONSE TO VIBRATION

The objective of vibration testing is to relate defects in the structure to changes in its dynamic characteristics. Vibration methods carried out over a period of time measure loss of stiffness and not loss of strength. Although a loss of stiffness implies a loss of strength, a serious loss of strength in an individual member may occur before there is a measurable loss of stiffness in the overall structure (*3*).

The use of vibration testing to indicate defects has been investigated in Britain (*155, 161, 162*). One technique involves temporarily attaching accelerometers to the structure and recording traffic and wind-induced vibrations. The modes of vibration and damping may then be determined by computer analysis and this provides a signature of the structure that will only change if the properties of the structure or its supports are changed. A somewhat different technique involves the application of a variable frequency sinusoidal force at a point in the structure and the measurement of responses at other points. These responses depend mainly on fixity and the stiffness of connections. Methods of interpreting the results in terms of type of defect and its cause have yet to be developed and these will inevitably have to be empirical. Nevertheless, vibration methods are sufficiently sensitive to show promise and may develop into a useful diagnostic technique (*162*).

CHAPTER SEVEN

FUTURE RESEARCH

INTRODUCTION

Many of the future research needs follow logically from the identification of the disadvantages of the existing techniques in Chapters 3 to 6. The scientific principles on which the various methods of detecting deterioration are based are well known and summarized in Chapter 2, and it is therefore unrealistic to anticipate entirely new methods of investigating the condition of highway structures. Because the simple test procedures, such as striking the surface of a component, have been thoroughly investigated, the majority of future developments will result from measurements of penetrating or reflected waveforms or magnetic disturbances. It is to be expected that advances will have to be made through refinements to existing equipment or the application of technology from other fields. Infrared thermography from heat-loss studies and ground penetrating radar from military applications are good examples of the latter. As electronic circuitry becomes more advanced, this will lead to equipment that is more discriminating, rugged, portable, and less expensive.

Although there is a demand by highway agencies for test methods that are rapid (because of minimum disruption to traffic) and that automatically reduce the data to common civil engineering terms, high technology has not been embraced by the highway industry to the same degree as it has been by most other industries. The major implication of this situation is that if new developments are to have a significant impact, the method must be compatible with the operator expertise commonly available in highway agencies or the service must be readily available from specialized contractors. It also means that some of the more sophisticated and expensive techniques already available, such as computerized tomography, are unlikely to become standard operational procedures and will be used only for special applications.

For the purposes of discussion, it is convenient to divide the identification of research needs into those that are common to all structures and those that specifically apply to concrete, steel, or timber structures. In identifying these needs, the approach taken has been to identify topics having the highest priority rather than to create a comprehensive list of researchable subjects.

ALL STRUCTURES

1. Perhaps the greatest need in the immediate future is not for the development of new techniques but for an objective assessment of the capabilities of existing techniques. There have been significant advances in both equipment and applications in recent years but many of these developments are unknown to engineers having operational responsibility. Field trials have rarely been coordinated between different agencies. Much of the experience has either not been reported, especially if unsuccessful, or has been the subject of exaggerated claims. Consequently there is an immediate need for highway agencies to use

these new techniques in the field under controlled conditions and report the results obtained.

2. Techniques that examine the overall response of a structure to an external stimulus, such as load or vibration, offer considerable scope for further development. Many of the traditional methods are capable of detecting defects or deterioration in individual components but the problem of predicting the effect of the deterioration on the performance of the structure remains. This is particularly true in highly redundant structures or in concrete and timber structures because the strength of concrete and timber is more susceptible to change during the life of a structure than is the strength of steel. Measurement of the overall response of a structure to known forces enables both strength-enhancing features, such as composite action or the stiffening effect of sidewalks and parapet walls, and capacity-reducing effects, such as loss of prestress or deteriorated connections, to be quantified. The results can then be used to determine the load-carrying capacity or, if repeated at regular intervals, changes in bridge performance with time.

3. New applications of proven technology may have a significant impact on the methods used to investigate the condition of highway structures. Two examples that appear worthy of investigation are the use of magnetic methods to detect corrosion currents in reinforced and prestressed concrete and radar to detect voids and differences in the density in timber components.

4. At the operational level, there is a need to match the capabilities and cost of the many investigative techniques available to the utilization of the results. In other words, a visual inspection may suffice to determine repair priorities but a delamination survey and coring would be needed to justify deck replacement. To satisfy this need, better data on the accuracy and cost of each procedure would be needed. However, the final result would be the ability to base operational decisions on reliable data generated at the least cost.

5. Although activities at the time of construction are not strictly within the scope of this report, there are many occasions when investigations would be much easier or less expensive if certain features had been included in the original design. Typical examples range from access to the interior of closed cells to the provision of electrical contact points to the reinforcement. Consequently there is a need to identify inexpensive improvements that could be made to new bridge designs and that would facilitate inspection and rehabilitation.

CONCRETE STRUCTURES

1. A nondestructive method is required to measure the rate of corrosion of steel in concrete. Existing half-cell tests measure the presence of corrosion but give no indication of the rate of the corrosion reactions. The ability to determine rate of corrosion is needed to predict the remaining service life of a structure and to monitor the effectiveness of repair techniques. Linear polarization techniques have shown some promise but the procedures are complex and instrumentation must be developed that is suitable for field use. Another procedure, AC impedance, although not as well developed as linear polarization, may prove to be more satisfactory and needs to be thoroughly investigated.

2. Other needs that are related to predicting the susceptibility of a concrete component to the corrosion of embedded reinforcement are techniques for measuring the depth to which chloride ion has penetrated, and the permeability of the concrete, its moisture content, and its resistivity.

3. Highway agencies have an immediate need for equipment that will investigate the conditions of prestressed, but especially post-tensioned, concrete. The use of radiographic techniques in France are encouraging and there is an urgent need in North America to build on the European experience to develop a viable method for detecting defects in grout and strands that are embedded in metal ducts.

4. Rapid, remote-sensing techniques based on radar and infrared thermography have been found to be powerful tools for detecting hidden deterioration in concrete. Further developments of these techniques such that standard test procedures can be formulated appears justifiable.

STEEL STRUCTURES

1. Evaluating the remaining fatigue life of existing steel bridges is a serious problem. Many bridges were built before the present AASHTO requirements for the design of new bridges were developed. Furthermore, these provisions are not directly applicable to specific existing bridges. An experimental technique, possibly based on the testing of coupons, is needed to complement analytical studies to predict remaining life expectancy.

2. Techniques for detecting corrosion in eye-bars and cables such as are used in cable-stayed and suspension bridges are limited and significant improvements are needed for the inspection of these critical components.

3. A study is needed to define the test techniques applicable to those components of steel structures that require special attention because of their critical nature, the difficulty of inspection, or hangers, eye-bars, and other pin connections where corrosion not only causes a reduction in cross section but corrosion products can distort members and induce stress concentrations. Other components where corrosion products can induce out-of-place distortion that could produce fatigue cracking also need to be identified.

4. Significant advances have been made in the application of ultrasonic methods to the detection and identification of flaws in steel components. Continued development is needed to improve characterization and to automate data processing to reduce the level of skill required by the operator.

TIMBER STRUCTURES

Because timber bridges are relatively few in number, tend to be smaller than the average bridge size, are usually located on secondary highways, and are more amenable to strengthening or the replacement of selected components, the opportunities for cost-effective research into investigative techniques are fewer than for steel or concrete structures. Specific subject areas that are worthy of future study are:

1. The strength of timber used in construction not only varies widely but timber is also susceptible to changes in strength in service. Consequently the greatest need is for methods that will measure the in-place strength (rather than stiffness) for evaluation of load-carrying capacity. In this respect energy-decay techniques, based on either impact or ultrasonic excitations, hold

promise. Any other technique that can be used to measure the modulus of elasticity is also worthy of investigation because modulus of elasticity has been used successfully as a predictor for strength in virgin timber.

2. Rapid, nondestructive techniques are needed for determining the presence of voids and differences in moisture content and density. Such measurements are possible using existing techniques but tend to be labor intensive and consequently expensive.

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Appendix A

SURVEY OF CURRENT PRACTICE

To determine the current state of the practice in detecting defects and deterioration in highway structures, a survey form (Fig. A-1) was mailed in October 1983. Replies were received from all 50 states, the District of Columbia, Puerto Rico, Guam and three Canadian Provinces. The responses are summarized in Table A-1 and observations on the responses are given below.

CONCRETE COMPONENTS

Concrete components are being tested much more extensively than either steel or timber components. This is, to a large extent, because of major bridge-deck rehabilitation programs underway in most states. It is common practice to carry out a condition survey of the deck so that the results may be used to assist in determining the method of repair and in preparing the contract documents. The widespread use of test procedures on concrete components also reflects the availability of several tests that are simple and inexpensive but that yield meaningful results.

Many states did not indicate that visual inspection tests were used routinely although it is known that visual examinations are made every two years. This is also true of steel and timber structures. There was also some confusion surrounding the determination of chloride-ion content in-situ. The question was intended to determine those jurisdictions that actually measure the chloride-ion content at the structure. Most of the affirmative responses appear to indicate that pulverized samples are taken in the field but the actual chloride determination is made in the laboratory using wet chemical analysis. Very few states are using polarization techniques, which is indicative of the relative infancy of these procedures. Some of the positive responses to this question may also have confused polarization techniques with potential measurements.

The widespread use of cores to determine properties of concrete reflects the ease with which samples of concrete can be removed and the widespread availability of test methods that are relatively inexpensive.

STEEL COMPONENTS

Except for visual examination, there is little routine testing of steel components. Many states commented that although a test method was shown as being used "sometimes," its use was very infrequent. The most common test method used in the field is the measurement of member thickness to determine section loss. Of the methods available for detecting flaws, ultrasonics and penetrating dyes are those in most common use. X-ray radiography is slightly more popular than gamma-ray radiography. Although several states reported that magnetic particle methods are sometimes used, only one state (Maryland) reported the use of eddy currents.

The removal of steel sections for laboratory testing is done very infrequently and usually to investigate a specific problem.

1. REINFORCED AND PRESTRESSED CONCRETE STRUCTURES

	Routinely	Sometimes	Never	Test Method
In-situ Testing Visual Inspection(1) (1)				
Delamination				
Concrete Cover				
Chloride Ion				
Corrosion Potentials				
Polarization Tests				
Other(2) (2)				
Cores Strength				
Air-void System				
Chloride Ion				
Petrographic				
Other (2)				

2. STEEL AND WROUGHT IRON STRUCTURES

	Routinely	Sometimes	Never	Test Method
In-Situ Testing Visual Inspection(1) (1)				
Ultrasonic				
Thickness				
Penetrating Dyes				
Magnetic Particles				
Eddy Currents				
X Rays				
γ Rays				
Film Gage (Coatings)				
Other(2) (2)				
Coupons Strength				
Hardness				
Ultrasonic				
Thickness				
Penetrating Dyes				
Magnetic Particles				
Eddy Currents				
X Rays				
γ Rays				
Film Gage (Coatings)				
Other(2) (2)				

FIGURE A-1 Survey form mailed October 1983.

3. TIMBER STRUCTURES

	Routinely	Sometimes	Never	Test Method
In-Situ Testing Visual Inspection(1) (1)				
Ice pick or Nail				
Increment Borer				
Other(2) (2)				
Cut Samples Strength				
Fungi				
Vermin				
Weathering				
Other(2) (2)				

Explanatory Notes (frequency, experience, end results, manuals, research-in-progress)

Footnotes
1. Insert what kind of defects are recorded
2. Insert other procedures used

FIGURE A-1 Survey form mailed October 1983 (*continued*).

TIMBER COMPONENTS

Several states commented that there are few timber bridges under their jurisdiction. The testing of these structures tends to be relatively unsophisticated. Twelve states indicated that a sounding hammer or similar device is used to detect decay, sometimes in preference to the use of an increment borer or a nail or ice pick. Electric drills are also used occasionally for this purpose.

Samples are rarely cut from timber for laboratory testing. Even where positive responses were received, it is suspected that the presence of fungi, vermin, and weathering may indicate that these observations are made in the field rather than on cut samples in the laboratory.

SURVEY: DETECTING DEFECTS AND DETERIORATION IN HIGHWAY STRUCTURES

The purpose of this survey is to determine which test methods are in use to detect defects and deterioration in concrete, steel, wrought-iron, and timber structures. It is also the intent to find out under what circumstances the tests are used, the degree of success achieved, and the manner in which the results are used.

The survey sheet (Fig. A-1) has been designed to allow the maximum flexibility in responses. For each type of structure, the test methods are separated into those carried out in the field and those performed on specimens removed from the structure. Wherever possible, manuals, reports, and research-in-progress should be identified.

TABLE A-1

SURVEY OF CURRENT PRACTICE (1983)*

State, District, Province or Territory	Reinforced and Prestressed Concrete Structures												Steel and Wrought Iron Structures																					Timber Structures								
	In-Situ							Cores					In-Situ										Coupons											In-Situ				Cut Samples				
	Visual	Delamination	Cover	Chloride Ion	Corrosion Potential	Polarization	Other	Strength	Air Voids	Chloride Ion	Petrographic	Other	Visual	Ultrasonic	Thickness	Penetrating Dyes	Magnetic Particles	Eddy Currents	X Rays	Gamma Rays	Film Gage	Other	Strength	Hardness	Ultrasonic	Thickness	Penetrating Dyes	Magnetic Particles	Eddy Currents	X Rays	Gamma Rays	Film Gage	Other	Visual	Ice Pick or Nail	Increment Borer	Other	Strength	Fungi	Vermin	Weathering	Other
Alabama		S	S	N	S	N	S[a]	S	N	S	N	S[b]		R	R	S	N	N	N	N	N		N	N	N	N	N	N	N	N	N	N			S	S	S[c]	N	N	N	N	
Alaska	R	S	S	S	S			S		S			R				S				S															S	R[c]					
Arizona		S	S	S	S	N		S	N	S	N			S		S			S	S															N	S		N	N	N	N	
Arkansas	R	R	N	N	N	N		S	N	N	N		R	S	R	S	N	N	N	N	S		S	S	N	S	N	N	N	N	N	N		R	R	R		S	N	N	N	
California		R	R	N	S	N		S	S	S	S	S[d]	R	S	S	S	S	N	S	S	N	S[e]	S	S	N	N	N	N	N	N	N	N	S[f]	R	N	R	S[c]	S	R	S	R	
Colorado	R	S	S	S	S	N		S	N	S	N			N	S	N	N	N	N	N	N		N	N	N	N	N	N	N	N	N	N		R	N	S	R[c]	N	N	N	N	
Connecticut		R	S	N	R	N		S	N	S	N			S	S	S	S	N	N	N	S		S	S	N	N	N	N	N	N	N	N	S[g]		R	S		N	N	N	N	
Delaware	R	R	R	R	R			S		S			R	S	S	S	S	N	N	N	R												S[g]	R	R	S						
Florida	R	R	R	N	R	S		R	N	R	N		R	S	R	S	S	N	S	S	S		N	N	N	N	N	N	N	N	N	N		R	R	R			R	R		
Georgia	R	S	R	S	S	N	S[ah]	R	R	S	N	S[ij]	R	R	S	S	R	N	N	S	R		R	S	S	R	S	S	N	N	N	N		R	S	R	S[g]	S	S	N	S	
Hawaii		S	S					S													S														S				S		S	
Idaho		R	R	S	S	N		S	N	S	N			S	R	N	N	N	N	N	S		S	S	S	S	S	S	S	S	S	S		R	N	S						
Illinois	R	R	S	S	N	N		S	N	S	N		R	S	N	S	N	N	N	N	N	S[e]	N	N	N	N	N	N	N	N	N	N	S[f]	No timber structures								
Indiana	R	S	S	S	S	N	R[a]	S	S	S	S		R	S	N	S	N	N	N	N	R		S	S	S	N	N	N	N	N	N	N	S[f]	R	R	N	R[c]	N	N	N	N	
Iowa	R	S	S		S	N		S	S	S	S			S	S	S	N	N	N		S		S	S	S	S	S		N	N		S				S		N	N	N	N	
Kansas	R	R	S	R	R	N	S[aho]	S	S	R	S	S[kl]	R	S		S			S															R								
Kentucky		R	R	S	S			S		S				S		S	S		S					S	S	S			S	S	S		S		R	S	R[c]	N	N	N	N	
Louisiana		R	S	S	R			S		S				S	R	S	S		S		R		S	S	S	S	S	S	S	S		S		R	R	R	R[c]	N	R	R	R	
Maine	R	R	R	R	R			R	N	R	N		R	S	S	S	N	N	N	N	R		S	S	N	S	S	N	N	N	N	N		R	R	S		N	S	S	N	
Maryland	R	R	S	R	R		S[m]	S	S	S	S		R	R	R	R	N	R	R	R	R		S	S	S	S	S	N	S	S	S	S	S[g]	R	R	S	R[ch]				S[n]	
Massachusetts		R	S	S	S	N		S	S	S	N			S	R	S	N	N	N	N	S			S										R	R	R			S	S		
Michigan	R	R	R	R	S	N	S[aho]	S	S	S	S	S[i]	R	S	S	S	S	N	R	S	N	S[p]	S	S	N	S	S	N	N	N	N	N	S[q]	S	S	S		S	N	N	N	S[r]
Minnesota		R	N		S	N		R	S	R	S										R		R			R										R						R[rs]
Mississippi		S	S	N	N	N		S	N	N	N			N	N	N	N	N	N	N	S		N	N	N	N	N	N	N	N	N	N			S	S		N	N	N	N	
Missouri	R	R	R	N	R	S		S	S	S	N		R	S	S	S	S	N	N	N	S		N	N	N	N	N	N	N	N	N	N		R	R	S	R[c]					
Montana	R	S	S	S	S								R																					R		S		N				
Nebraska	R	R	R	N	R	N		R	N	R	N			S	R		R	N	R	R	R		R	R	S	R	N	N	N	N	N	S		R	S	S		N	S	S	R	R[r]
Nevada	R	S	S	N	N	N		S	N	S	N		R	S	S	S	S		N	S	S	N	N	S	S	S	S	N	N	N	N	N		R	R	S		N	S	S	R	
New Hampshire		S	S	S	S	N		S	N	R	N			N	N	S	S	N	S	N	S		N	N	N	N	N	N	N	N	N	N			S	S		N	N	N	N	
New Jersey		S	S	S	S	N		S	N	N	N			S	R	S	S	N	N	N	N		R	S	S	S	N	N	N	N	N	N	S[g]		R	S	S[s]	N	N	N	N	
New Mexico	R	S	S	S	S	N		S	N	S	N		R	N	N	S	S	N	R	N	S		S	S	S	S	S	N	N	N	N	N		R	R	S		N	N	N	N	
New York	R	R	R	N	R	N		R	R	S	R	R[t]	R	S	R	S	S	N	N	S	S	S[u]	S	S	N	N	S	S	N	S	S	S	S[fq]	R	R	N		N	N	N	N	
North Carolina			S	S	S	N		R	S	S	N		S	S	S	S	S	N	N	R	R	R[v]	S	S	R	S	S				R	S	S[v]	S		R	S[c]		S	S	S	
North Dakota	R	S	S	S	S	N		S	N	N	N		R	N	S	S	N	N	N	N	S		S	S	R	R	R	R	R	R	R	R	S[g]	R	S	N		N	N	N	N	
Ohio		R	S	S	S	N		S	S	S	S	S[ao]		S	S	S	S	N	R	R	R		S	S	N	S	S	S	N	S	S	N	S[g]		R	S	R[c]	N	S	S	N	
Oklahoma	R	R	R	N	S	N		S	S	R	S	S[l]	R	S	N	S	S	N	S	N	N		N	N	N	N	N	N	N	N	N	N	S[p]	R	S	N		N	N	N	N	
Oregon	R	S	S	S	S	N		S	N	S	N		R	S	S	S	N	N	N	N	S													R	R	R						
Pennsylvania	R	S	S	S	S	N	S[a]	R	R	R	R		R	S	S	S	S	N		N			S	S			S	S				S			S	S						
Rhode Island	R	R	S	R	R			S					N																					N								
South Carolina	R	S	S	S	N	N		S	N	N	N		R	S		S	S		S		S		N	N	N	N	N	N	N	N	N	N		S	S	S		N	N	N	N	
South Dakota	R	R	R	R	R	N		S	N	S	N		R	S	S	S	S	N	S	N	N													R	S	S		N	N	N	N	
Tennessee	R	R	N	R	N			S	N	R	N			S	S	S	N	N	N	S	S													N	N	N		N	N	N	N	
Texas	R	R		S	S	S		S	S	S	S		R	S	S	S	N	N	N	S	S		S	S	N	S	N	N	N	N	S	S		R	S	S	S[c]					
Utah		R	S	S	S	S		S	S	S	S			R	R	S	S	N	S	S	R		R	S	S	S	S	S	S	S	S	S			N	R		N	N	N	N	
Vermont	R	R	R	R	R	N		S	S	S	N		R	N	R	S	N	N	N	N	S		N	N	N	N	N	N	N	N	N	N		R	R	S		N	N	N	N	
Virginia	R	S	S	S	S	N		S	S	S	S		R	S	N	S	N	N	N	S	N		S	S	N	N	N	N	N	N	N	N		R	S	S		N	N	N	N	
Washington	S	S	S	R	S			S					R	N	N	N	N	N	N	N	N		N	N	N	N	N	N	N	N	N	N		N	N	R		N	N	N	N	
West Virginia	R	R	R		R			R	R	R	R		S	S	N	S	S	N	N	S	N		N	N	N	N	N	N	N	N	N	N		S	N	N		N	N	N	N	
Wisconsin	R	R	R	S	S	N		S	S	S			R	S	S	S	S	N	S	N	S		S	S	S	S	S	S	N	S	N	S	S[g]	R	R	S		N	N	N	N	
Wyoming		R	R	R	R								R	S	S	S	S	N	N	N	S		S	N	N	S	N	N	N	N	N	N		S	R	S		N	S	N	S	S[r]
District of Columbia		S	N	S	S	N		N	N	S	N			S	N	S		N	N	N	N			N	N	N	N	N	N	N	N	N		No timber structures								
Puerto Rico	R												R																					No timber structures								
Guam	N	N	N	N	N	N		N	N	N	N		No steel structures																					No timber structures								
Nova Scotia	R	R	R	R	R	N		S	N	R	N		R	N	N	N	N	N	N	N	N		N	N	N	N	N	N	N	N	N	N		R	S	N		N	N	N	N	
Ontario	R	R	R	R	R	N		R	R	R	S		R	S	S	S	N	N	S	N	S		S	S	N	S	N	N	N	N	N	S	S[g]	R	S	S		S	S	S	S	
Saskatchewan	R	S	S	S	R	N		S	N	N	N		R	S	S	S	N	N	S	N	S		N	N	N	N	N	N	N	N	N	N		R	S	N		N	N	N	N	

*R = Routinely; S = Sometimes; N = Never; Blank = No response provided

[a] Impact hammer
[b] Presence of delamination
[c] Sounding hammer
[d] Cement content
[e] Strain gages
[f] Impact resistance (Charpy)
[g] Chemical composition
[h] Ultrasonic testing
[i] Unit weight and absorption
[j] Splitting tensile strength
[k] Permeability
[l] Dye penetration
[m] Electrical resistance
[n] Increment borer
[o] Windsor Probe
[p] Etching
[q] Metallographic examination
[r] Preservative content
[s] Moisture content
[t] Freeze-thaw resistance
[u] In-situ hardness
[v] Coating adhesion

Appendix B

STRESS WAVES IN SOLID MEDIA

When vibration is propagated through a solid medium, three kinds of waves are generated, namely longitudinal, transverse, and surface waves. These three waves travel at different speeds. The longitudinal or compressional waves travel about twice as fast as the other two types and the particles of the medium move in the same direction as the wave being propagated. The transverse or shear waves are the next fastest and have particle displacement at right angles to the direction of travel. The surface or Rayleigh waves resemble water waves and are the slowest of the three. These travel along the surface and the amplitude decreases with depth. The individual particles travel in elliptical orbits. The velocities of the stress waves through a solid material are a function of its density and elastic constraints.

For the compression wave

$$V_c^2 = \frac{E(1-\mu)}{\rho(1+\mu)(1-2\mu)} \tag{B-1}$$

For the transverse wave

$$V_t^2 = \frac{E}{2\rho(1+\mu)} = \frac{G}{\rho} \tag{B-2}$$

where

V_c = velocity of the compression wave,
V_t = velocity of the transverse wave,
E = dynamic elastic modulus,
G = shear modulus,
μ = Poisson's ratio, and
ρ = density.

For the surface (Rayleigh) wave

$$V_R = AV_t \tag{B-3}$$

where A is found from

$$A^6 - 8A^4 + 8(3 - 2Y^2)A^2 - 16(1 - Y^2) = 0 \tag{B-4}$$

in which

$$Y = \frac{V_t}{V_c} = \frac{1-2\mu}{2(1+\mu)} \tag{B-5}$$

Thus the ratios V_t/V_c and V_R/V_c depend only on Poisson's ratio for the material. These ratios have been plotted for different volumes of Poisson's ratio in Figure B-1. Equation B-1 is sometimes simplified to

$$V_c = \sqrt{\frac{E}{\rho}} \tag{B-6}$$

which is the same as Eq. 2 in Chapter 2. This simplification is permissible because Eq. B-1 is relatively insensitive to changes in Poisson's ratio (*31*). Furthermore, pulse-velocity techniques usually involve the comparison of velocity readings from one part of a component to another rather than establishing absolute values.

Longitudinal waves are normally used in pulse velocity measurements because their transit time is easily detected as there is no interference from the slower transverse and surface waves.

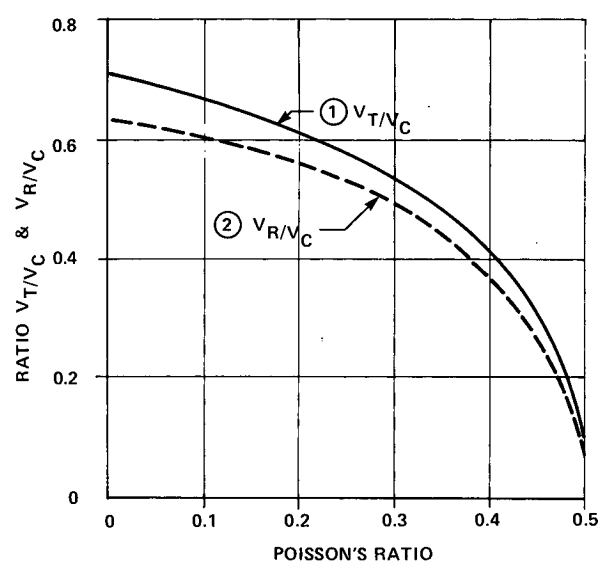

FIGURE B-1 Theoretical relationships between stress wave velocities and Poisson's ratio.

Appendix C

THE FOUR-PIN METHOD OF RESISTIVITY MEASUREMENT

Consider the distribution of potential in a homogeneous, semi-infinite material of resistivity ρ at a point electrode, P (Fig. C-1). If the current entering the ground is $+I$, the potential difference across a shell dr thick, distance r from P is

$$\text{since } V = IR = \frac{I\rho\ell}{a}$$

$$dV = -I\rho \frac{dr}{2\pi r^2}$$

and the potential at a distance r from the current source P is

$$V(r) = \frac{I\rho}{2\pi}\frac{1}{r}$$

The potential at any given point is then $V = V(r) - V(r^1)$ where r and r^1 are the distances from the positive and negative electrodes respectively (Fig. C-2).

If current is introduced at the two electrodes c_1 and c_2 and the potential difference is measured between as points P_1 and P_2

$$\text{Potential at } P_1 = \frac{I\rho}{2\pi}\left(\frac{1}{r_1} - \frac{1}{r_2}\right)$$

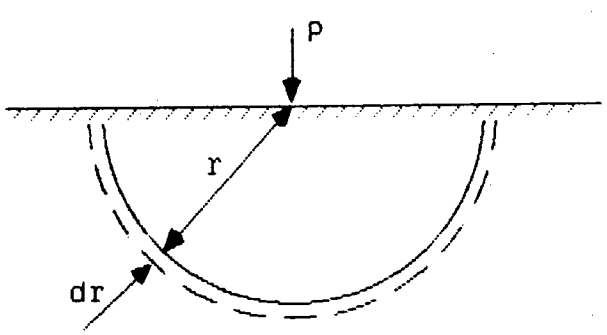

FIGURE C-1

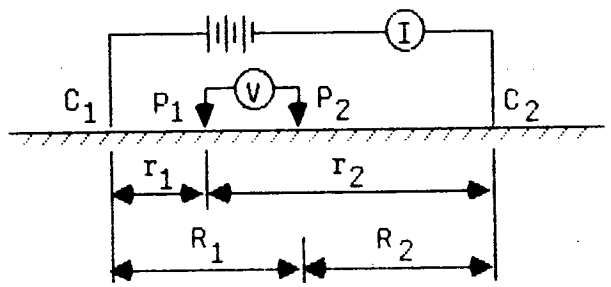

FIGURE C-2

$$\text{Potential at } P_2 = \frac{I\rho}{2\pi}\left(\frac{1}{R_1} - \frac{1}{R_2}\right)$$

And the potential difference, E is $E_{P_1} - E_{P_2}$

$$E = \frac{I\rho}{2\pi}\left(\frac{1}{r_1} - \frac{1}{r_2}\right) - \frac{I\rho}{2\pi}\left(\frac{1}{R_1} - \frac{1}{R_2}\right)$$

$$= \frac{I\rho}{2\pi}\left(\frac{1}{r_1} - \frac{1}{r_2} - \frac{1}{R_1} + \frac{1}{R_2}\right)$$

Then

$$\rho = 2\pi\frac{E}{I}\left(\frac{1}{\frac{1}{r_1} - \frac{1}{r_2} - \frac{1}{R_1} + \frac{1}{R_2}}\right)$$

But in the four pin method (see Figure 5)

$$r_1 = R_2 = a,\ r_2 = R_1 = 2a$$

$$\rho = 2\pi\frac{E}{I}\left(\frac{1}{\frac{1}{a} - \frac{1}{2a} - \frac{1}{2a} + \frac{1}{a}}\right)$$

$$\rho = 2\pi\frac{aE}{I}$$

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