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August 25-27, 1992

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John A. Volpe National
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K.H. Frank, N.P. Jones, B.R. Ellingwood, R.J. Ross, R.F. Pellerin, J.A. Plecknik, O. Henriquez, C.M. Teller, A.G. Davis, and E.G. Hennecke, II

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U.S. Department of Transportation
Federal Highway Administration
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, VA 22101

U.S. Department of Transportation
Research and Special Programs Administration
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55 Broadway
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On August 25-27, 1992, a major forum of 80 knowledgeable individuals representing researchers, manufacturers, and users of nondestructive evaluation (NDE) methods and equipment for bridges was convened to exchange information and to provide guidance for studies of NDE of bridges proposed as a part of the Federal Highway Administration's (FHWA) High Priority Area (HPA). The conference (1) identified the current status of NDE for bridges, (2) defined goals and areas of concentration for NDE research as it applies to bridge inspection, and (3) fulfilled an immediate need to inform NDE system developers of bridge inspection needs and the NDE system users of potential technologies.

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PREFACE

The Conference on Nondestructive Evaluation of Bridges was held in Arlington, Virginia on August 25-27, 1992. Thomas J. Pasko, Jr., of the Federal Highway Administration (FHWA), opened the conference. The Chairman of the conference was Robert E. Green, Jr., Johns Hopkins University. On the opening day, papers were presented during two sessions.

During the Plenary Session on Structural Problems, four persons presented papers:

- Karl H. Frank, University of Texas: Steel Bridges
- Nicholas P. Jones, Johns Hopkins University: Concrete Bridges
- Michael A. Ritter, U.S. Department of Agriculture, Forest Products Research Laboratory: Timber Bridges
- Joesph A. Plecknik, California State University - Long Beach: Composite Bridges

During the Plenary Session on State-of-the-Art Techniques, four persons presented papers:

- Cecil M. Teller, Southwest Research Institute: Steel Bridges
- Allen G. Davis, STS Consultants Ltd.: Concrete Bridges
- Robert J. Ross, U.S. Department of Agriculture, Forest Products Research Laboratory: Timber Bridges
- Edmund G. Henneke, II, Virginia Polytechnic Institute: Composite Bridges

The presenters of the Plenary Session on State-of-the-Art Techniques also led workshops in their respective areas on the second day, and they presented the findings of the conference on the third day. The conference was closed by the Chairman, Robert E. Green, Jr., and Charles H. McGogney (FHWA), the organizer of the conference.

This document contains the papers presented during the two plenary sessions. A complete transcript of the proceedings for August 25 and 27 is available from the National Technical Information Service (NTIS) on diskette as a separate document (FHWA-RD-93-040B/DOT-VNTSC-FHWA-93-2).

METRIC/ENGLISH CONVERSION FACTORS

ENGLISH TO METRIC

LENGTH (APPROXIMATE)

1 inch (in) = 2.5 centimeters (cm)
1 foot (ft) = 30 centimeters (cm)
1 yard (yd) = 0.9 meter (m)
1 mile (mi) = 1.6 kilometers (km)

AREA (APPROXIMATE)

1 square inch (sq in, in^2 = 6.5 square centimeters (cm^2)
1 square foot (sq ft, ft^2 = 0.09 square meter (m_2)
1 square yard (sq yd, yd^2) = 0.8 square meter (m^2)
1 square mile (sq mi, mi^2) = 2.6 square kilometers (km^2)
1 acre = 0.4 hectares (he) = 4,000 square meters (m^2)

MASS - WEIGHT (APPROXIMATE)

1 ounce (oz) = 28 grams (gr)
1 pound (lb) = .45 kilogram (kg)
1 short ton = 2,000 pounds (lb) = 0.9 tonne (t)

VOLUME (APPROXIMATE)

1 teaspoon (tsp) = 5 milliliters (ml)
1 tablespoon (tbsp) = 15 milliliters (ml)
1 fluid ounce (fl oz) = 30 milliliters (ml)
1 cup (c) = 0.24 liter (1)
1 pint (pt) = 0.47 liter (1)
1 quart (qt) = 0.96 liter (1)
1 gallon (gal) = 3.8 liters (1)
1 cubic foot (cu ft, ft^3) = 0.03 cubic meter (m^3)
1 cubic yard (cu yd, yd^3) = 0.76 cubic meter (m^3)

TEMPERATURE (EXACT)

$[(x-32)(5/9)]\ ^{\circ}F = y\ ^{\circ}C$

METRIC TO ENGLISH

LENGTH (APPROXIMATE)

1 millimeter (mm) = 0.04 inch (in)
1 centimeter (cm) = 0.4 inch (in)
1 meter (m) = 3.3 feet (ft)
1 meter (m) = 1.1 yards (yd)
1 kilometer (km) = 0.6 mile (mi)

AREA (APPROXIMATE)

1 square centimeter (cm^2) = 0.16 square inch (sq in, in^2)
1 square meter (m^2) = 1.2 square yeards (sq yd, yd^2)
1 square kilometer (km^2) = 0.4 square mile (sq mi, mi^2)
1 hectare (he) = 10,000 square meters (m^2) = 2.5 acres

MASS - WEIGHT (APPROXIMATE)

1 gram (gr) = 0.036 ounce (oz)
1 kilogram (kg) = 2.2 pounds (lb)
1 tonne (t) = 1,000 kilograms (kg) = 1.1 short tons

VOLUME (APPROXIMATE)

1 milliliters (ml) = 0.03 fluid ounce (fl oz)
1 liter (1) = 2.1 pints (pt)
1 liter (1) = 1.06 quarts (qt)
1 liter (1) = 0.26 gallon (gal)
1 cubic meter (m^3) = 36 cubic feet (cu ft, ft^3)
1 cubic meter (m^3) = 1.3 cubic yards (cu yd, yd^3)

TEMPERATURE (EXACT)

$[(9/5)\ y + 32]\ ^{\circ}C = x\ ^{\circ}F$

QUICK INCH-CENTIMETER LENGTH CONVERSION

INCHES	0	1	2	3	4	5	6	7	8	9	10

CENTIMETERS: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 25.40

QUICK FAHRENHEIT-CELSIUS TEMPERATURE CONVERSION

$^{\circ}F$	-40°	-22°	-4°	14°	32°	50°	68°	86°	104°	122°	140°	158°	176°	194°	212°
$^{\circ}C$	-40°	-30°	-20°	-10°	0°	10°	20°	30°	40°	50°	60°	70°	80°	90°	100°

For more exact and or other conversion factors, see NBS Miscellaneous Publication 286, Units of Weights and Measures. Price $2.50. SD Catalog No. C13 10286.

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EXECUTIVE SUMMARY

As of June 1988, 135,826 of the 577,710 bridges inventoried and classified in the United States were structurally deficient--i.e., were closed or required rehabilitation. These data were obtained by visual inspection. However, hidden, inaccessible, and internal anomalies that defy visual inspection can cause failure of individual bridge components and possibly the entire structure.

On August 25-27, 1992, a major forum of 80 knowledgeable individuals representing researchers, manufacturers, and users of nondestructive evaluation (NDE) methods and equipment for bridges was convened to exchange information and to provide guidance for studies of NDE of bridges proposed as a part of the Federal Highway Administration's (FHWA) high priority area (HPA). The conference (1) identified the current status of NDE for bridges, (2) defined goals and areas of concentration for NDE research as it applies to bridge inspection, and (3) fulfilled an immediate need to inform NDE system developers of bridge inspection needs and the NDE system users of potential technologies.

Four invited speakers described the requirements for NDE techniques for steel, concrete, timber, and composite bridges. Next, four other invited speakers discussed the current and future status of NDE for fulfilling these requirements. The conference participants formed four working groups: steel, concrete, timber, and composite construction. The second group of four speakers acted as facilitators in their respective areas. These groups formulated the recommendations of the conference.

Current Status of NDE for Bridges

It was made evident that civil structures, including bridges, have not been given high priority for the application of advanced NDE techniques as compared to aircraft and aerospace structures. In fact, visual inspection approximately once every two years is the primary technique used for routine inspection of bridges. Conventional NDE is used only as an adjunct to visual inspection, and very few advanced NDE techniques are used at all.

Unfortunately, there are many defects that are not visible on the surface and are internal, and therefore are not normally detected by visual inspection.

Steel Bridges

For steel bridges, cracks are the primary defects needing detection and sizing, with a special emphasis on welds. A method for monitoring the behavior of cracks under service conditions is needed. Corrosion of steel cables of suspension bridges is also a problem. Conventional radiography has been the method of choice for detection of volumetric defects in steel bridges. Other conventional NDE methods such as liquid penetrant, magnetic particle, eddy current, ultrasonic, and acoustic emission are being used for crack detection and monitoring. Each of these conventional NDE techniques is used as an adjunct to visual inspection.

Concrete Bridges

For concrete bridges, corrosion is the main problem needing detection, monitoring, and assessment. In particular, corrosion of steel rebars encased in concrete make inspection particularly difficult. The global techniques for inspection of concrete bridges are load testing and modal analysis, although the results of modal analysis techniques have proven rather limited in capability. The local techniques for inspection of concrete bridges are ultrasonic, pulse echo, magnetic, electrical resistivity, corrosion potential, infrared thermography, ground-penetrating radar, radiography, and acoustic emission.

Timber Bridges

The main cause of damage in timber bridges is attack by fungi, insects, and marine borers. There is currently no NDE methodology widely used for assessing the quality or the performance of timber bridges. Timber bridge inspection generally relies on primitive techniques such as destructive drilling or coring to locate deterioration. However, stress-wave techniques have been used in some applications and appear to be the NDE technique showing the most promise for timber bridge inspection.

Composite Bridges

Although currently there are no composite bridges, the types of defects expected are the same as have been observed in other composite structures subjected to mechanical loading. There are no NDE techniques developed specifically for composite bridges because no composite bridges exist, however, there are numerous relatively sophisticated NDE techniques for monitoring degradation of composite materials. These techniques have been primarily developed for the aircraft and aerospace industries.

Goals of NDE Research for Bridges

The participants of the conference reached consensus on the following points:

(1) The **primary anomalies to be addressed** by research are corrosion, cracks, and fracture of steel (exposed, painted, and embedded).

(2) The **characteristics of the equipment** to be enhanced or developed shall make that equipment easy to operate, portable, economical, shall provide a high probability of detection of flaws, and shall include equipment for local and global evaluations.

(3) **Analytical frameworks** that will provide or facilitate the interpretation of inspection data, will predict remaining life or needed inspection intervals, and will facilitate the decision processes (i.e., leave as it is, repair, retrofit, or replace) need to be extended or developed for bridge NDE.

(4) **Institutional issues** (such as development of NDE procedures, standards, and training, and incorporation of NDE requirements in bridge designs) must be addressed.

The specific guidance for NDE research for bridges, as concluded from the conference, includes:

- Develop inexpensive, rapid, automated inspection techniques.
- Develop techniques that provide quantitative flaw-size information not subject to operator interpretation.
- Create continuing education programs for engineers involved in bridge construction and maintenance.
- Develop multimode, multisensor NDE techniques that will yield complementary information about anomalies.
- Integrate the sensitivity of inspection methods and severity of the discontinuity into realistic accept/reject criteria.
- Develop standardization and codes of practice.
- Develop self-monitoring systems, including sensors and actuators.
- Plan for NDE as part of the design process for new bridges.

Although no composite bridges currently exist, the general feeling was that planning for NDE of composite bridge structures should begin now, since such structures afford outstanding opportunities for embedded sensors and actuators. The participants felt that local NDE inspection techniques need improvement and that reliable global NDE techniques need development. It should be noted that emphasis given to institutional problems was greater than initially anticipated.

NONDESTRUCTIVE EVALUATION PROBLEMS IN STEEL BRIDGES

Karl H. Frank
University of Texas at Austin
Department of Engineering
Austin, TX

ABSTRACT

This paper describes some of the problems in steel bridge construction and field inspection which can be solved through improved nondestructive inspection. The state of present fabrication and in-service inspection is outlined. Three examples of failures of present technology to solve or to cause problems are presented. New technology, improvement of existing technology, or automation are needed to provide more reliable and less costly structures. Coupled with the improvements in inspection technology, new acceptance criteria must be developed which allows the improvements in inspection to be translated into more rational acceptance and evaluation procedures. The inspection equipment and procedures must be able of producing quantitative information of flaw size and location. Technologies that require subjective evaluations are not reliable enough to insure the safety of our bridges.

1. INTRODUCTION

The inspection problems associated with the construction and maintenance of steel bridges are summarized in this paper. Numerous opportunities exist for improvements in the safety and reduction in costs through the use of enhanced nondestructive inspection. In order for the improvements in inspection to be translated into practice, the technology must be reliable, easy to use in both the shop and field environment, and provide quantitative information. Bridge inspection technology has not progressed as rapidly as in other industries, primarily due to lack of funding. A coherent and well planned program of research and technology transfer is needed.

This paper presents some classic inspection problems that have occurred in the bridge industry. These are presented to focus upon the shortcomings of the present inspection methods and to provide basis to begin a discussion of improved inspection technology to circumvent these problems in the future.

A summary of present inspection procedures is presented. The need for improvements are cited. The inspection needs are divided into shop or fabrication inspection and field inspection. Present shop inspection procedures and criteria are based on historical rather than engineering judgement. Improved shop inspection techniques along with acceptance criteria based upon fitness for purposed will result in more economical and safe structures. Reliable field inspections are necessary as our structures age in order to assess their safety. At the present, the major cracking problems in steel bridges are associated with cracking at connections of transverse members to the main longitudinal structure. These cracks are caused by a lack of proper detailing and failure to account for the three dimensional behavior of the bridge in the design analysis. These cracks are an expensive nuisance. They are not threatening the safety of the structure since they will normally either arrest or grow at a decreasing rate as they extend. However, when an existing bridge is evaluated for increase in traffic or to determine its suitability for future use, the inspection of butt or groove welds is often performed even though no problems are evident. Often anomalies in the welds are found. These discontinuities are often not new cracks generated from service conditions, rather they are defects originally accepted by the inspection used during fabrication. We need a reliable method to assess the significance of these discontinuities.

2. NONDESTRUCTIVE INSPECTION FAILURES

The most often cited example of a bridge failure due to fracture is the collapse of the Pt. Pleasant Bridge spanning the Ohio River at Pt. Pleasant, West Virginia. The collapse occurred due to the unstable extension of cracks at the inside of a pin hole in one of the eye-bars forming the suspension chain of the bridge. The eye-bar material was a heat treated 1060 steel with very poor fracture toughness. The crack that initiated the failure was approximately 1/8 in. deep. After the cause of the failure was determined, a companion bridge on the Ohio River near St. Marys with the same eye-bars was inspected to determine if it contained similar cracks. A detailed ultrasonic inspection of the eye-bars was unable to determine whether cracks of the order of 1/8 in. in depth existed in the St. Marys Bridge. The rough bore of the pin holes and the corrosion in the pin connection made it impossible to resolve small cracks in these large members. Since it was impossible to insure that the eye-bars in the St. Marys Bridge did not contain these small cracks, it was dismantled. An improved inspection method able to detect and size the small cracks would have allowed consideration of other strategies to save the bridge.

The collapse of the Pt. Pleasant Bridge led to the start of the national bridge inspection program and the implementation of toughness requirements for bridge steels. The toughness requirements insure that the stable crack size in bridge members will be of the order of 1 to 2 inches at the lowest operating temperatures and full design stress. This higher toughness steel with its corresponding larger critical crack size eliminates the need to detect the very small cracks required in the inspection of the St. Marys Bridge. The large crack size should be easily detectable with conventional technology. Problems of inspection still exist. We need a rapid and easy to use inspection procedure that provides assurance that we can find and repair crack fatigue cracks before they reach their critical size.

Two bridges where fabrication NDE deficiencies have led to fracture problems are the I-79 Bridge in Pittsburgh and the Pt. Pleasant Memorial Bridge (the replacement for the bridge that collapsed). The I-79 bridge suffered a fracture of the tension flange and web of one of its girders. The bridge was in service at the time of this fracture and traffic continued across the bridge with this fractured girder. The cause of the early fracture of the bridge, it had been in service less than a year, was a defect in a weld repair. The flange splices in the bridge were made using the electroslag welding process. During the welding of the flanges, the process was interrupted to mount new coils of welding wire. The interruption caused a lack-of-fusion defect to occur in the weldment. After the weld was completed, the retaining shoes were removed and the obvious lack of fusion on the surface was repaired using shielded metal arc welding. The weldment was then radiographed. The radiograph revealed internal lack of fusion. This was removed by air arc gouging out the area. The gouged area was then filled by using an unknown welding process. The weld was radiographed again and no defects were found. The failure investigation of the bridge fracture revealed that a large defect located in the weld repair extended by fatigue and triggered a brittle fracture. The planar defect was of the order of 1 inch in diameter. This large defect escaped detection in the radiograph of the weld repair and caused the fracture of the bridge member. Obviously, the radiography was not adequate. Proper radiography techniques should be able to detect such large cracks. The reliability of radiography must be improved as we move towards more efficient structures which demand higher performance.

A 3-1/2 inch long crack was found during a visual inspection of the Pt. Pleasant Memorial Bridge. The crack was in a fracture critical tension member of high strength A514 steel. The crack was located at welded flange splice. The discovery prompted the closure of the bridge and an inspection of the other welds in the bridge. Many smaller defects were found in the other welds by ultrasonic inspection. The large 3-1/2 inch crack was located in an area of two weld repairs. A small defect was visible in the radiograph of the original weld. A weld repair was attempted and then radiographed. The defect was still visible in this second radiograph. A second weld repair was performed and the subsequent radiograph showed no defects. Fractographic analysis of the fracture surface did not reveal the existence of a preexisting defect. Evidently, the cracking occurred during the service life of the bridge due to the low toughness of the second weld repair metal. The small defect in the original weld would have likely not led to cracking. However, the attempts to remove this small defect with a weld repair produced a more critical condition. If the radiography acceptance standards were based on the members service condition, the small initial defect would have been allowed to remain. The cost of the two shop repairs and the in-service repair in the bridge could have been avoided. The other cracks found in the bridge were edge defects caused by improper extension of the weld into the run off tabs. These sometime large cracks, 1 inch long on the surface and up to 1/2 inch deep, were missed in the original radiography performed in the fabricating shop. Edge blocks were not used by the fabricator to shield the film and prevent backscattered radiation from exposing the film edge. The bridge was built by two fabricators. The welds made by the fabricator who used edge blocks did not contain these defects. Over thirty of the welds made by the second fabricator contained these cracks. Improper radiographic techniques caused welds with potentially critical defects to be accepted. A need exists for a fabrication inspection procedure which is reliable and can be used not only for rejecting weldments but also has the imaging ability to allow acceptance of discontinuities which meet fitness for purpose standards of acceptance.

3. FABRICATION INSPECTION

Inspection of steel bridges during fabrication is a crucial part of the fracture control of steel bridges. Most of the failures and problems associated with full penetration groove welds are traceable to inadequate fabrication inspection. Reliable methods of inspection that produce quantitative results are needed. The inspection methods must be low in cost, tolerant of the typical dust and heat of a fabricating shop, and capable of being used by normal shop quality assurance personnel. Rapid inspection is also a prime goal since long inspection times tie up shop and space and reduce productivity. In addition to inspection in the fabricating shop, some states use field welding. Inspection methods must be portable and able to be utilized efficiently in the field.

3.1 CRITICAL REVIEW OF CURRENT PRACTICE

The current fabrication inspection practice is to use either or both radiography and ultrasonic inspection for full penetration groove welds. Current radiography practice needs to be refined to provide reliable results. Use of edge blocks to prevent overexposure at edges must be incorporated into the specifications. Research is needed to develop proper procedures. The sensitivity and resolution of current practice needs to be determined so that acceptance and rejection criteria can be based upon performance requirements rather than workmanship standards. The ability to resolve various crack like defects in various plate thickness needs to be documented.

Ultrasonic inspection while most sensitive to planar crack like defects is a very tedious operation. The present methods of evaluating a reflector are difficult and led to different interpretations among operators. The development of an automated scanning and imaging system would be desirable. The inspection system should not only characterize the reflector in terms of reflected sound but more importantly give accurate size and location information. The goal should be a reasonably low cost, under $10,000, system that can scan welds from 10 to 40 inches in length of plates up to 4 inch in thickness. The equipment must be rugged, portable, and reliable. Inspection time should be less than 1 hour.

Partial penetration welds often used in box girders or in box members in trusses are a particular difficult inspection problem. Standard radiography cannot be employed due to the weld geometry. Ultrasonic inspection is difficult not only due to geometry but also in interpreting the intended lack of fusion. A simple means to evaluate these welds are needed. The welds are often very long of the order of hundreds of feet. An automated rapid inspection system is needed.

Magnetic particle is employed to evaluate fillet welds for transverse, longitudinal, and underbed cracking. This method is also sometimes employed along with the dye penetrant to inspect edges of groove welds. This is done manually usually over about 10 to 20% of the fillet weld. Fillet welds connecting webs to flanges, stiffeners to webs, and miscellaneous stiffening details are subject to this type of inspection. This inspection can take considerable time sometimes much longer then the time to produce the weld. An automated system is needed. The welding equipment employed for these welds is either semi-automatic or automatic. There appears to be no reason that automated inspection equipment cannot be developed.

All of the present fabrication inspection techniques are manual. They require considerable time. In the case of radiography, it must be done between normal working hours or an area of the plant must be shut down. The standards of acceptance and rejection are complex and subjective. A reliable and quick method of inspection which produces quantitative results will benefit the industry. The result would be lower costs due to less interruption of the work, repair of only critical defects, and a more reliable structure.

4. INSPECTION OF IN-SERVICE BRIDGES

Inspection of in-service bridges other than a visual inspection is normally done for one of the following reasons:

1. Determination of the performance of a structure not meeting present design specifications. This often occurs when a structure is to be widened or rehabilitated.
2. Cracking has occurred in another structure or in another place in the same structure. The inspection is performed to determine how wide spread is the problem.
3. After collision damage to the bridge has been repaired or area of cracking has been retrofitted. The inspection is performed to evaluate the effectiveness of the repair or retrofit.

In most cases a visual inspection is made followed by either an ultrasonic or dye penetrant inspection. In many of the cases of field cracking, the crack occurs at weld toe. Ultrasonic inspection has not been found to be a practical method for reliably finding weld toe cracks. A method to quickly find weld toe cracks is needed. The method should not only locate the crack but also determine the surface length of the crack and crack depth. Depth is important since the criticality of the crack can be judged from the depth rather than the length along the weld toe. The inspection method must be capable of finding cracks without the removal of paint. Paint removal is inexpensive and due to the use of lead base paints may require environmental protection methods. Paint thickness up to 30 mils may be encountered at weld toes in older bridges that have been repainted.

Butt or groove welds are sometimes inspected by ultrasonics when a bridge is being evaluated for its expected performance in the future. Many times the inspection discovers discontinuities that are unacceptable by current weld acceptance standards. Presently, the welds are cored to allow a destructive examination. This destructive examination is costly. An inspection and evaluation procedure needs to be developed to allow the performance of these welds to be accurately determined.

Field inspection is costly and access is often limited. Newer bridges often have access walkways or hand railings to allow inspection. However, inspectors must be capable of climbing and unafraid of the heights. One hundred percent inspection is too costly and not required in most cases. The inspection should be initially limited to the evaluation of only the critical and fatigue susceptible areas.

A method of monitoring the behavior of the crack under service conditions is also needed. The rate of crack extension is needed to determine the amount of time before a repair must be done. Also the same information is required when a cracked detail is retrofitted. The effectiveness of the retrofit to eliminate further crack extension needs to be determined. Due to the slow crack extension under fatigue loading, the monitoring system must be capable of gathering data unattended over a period of weeks or months.

5. SUMMARY

Improved nondestructive evaluation equipment and procedures will improve the safety of new and existing bridges. New inspection techniques along with realistic acceptance criteria will allow more economical fabrication and realistic assessment of in-service structures. The goal of the development of nondestructive inspection research should be to develop:

1. rapid and automated inspection techniques,
2. techniques that provide quantitative flaw size information which is reliable and is not subject to subjective operator interpretation,
3. and integration of the sensitivity of the inspection methods and severity of the discontinuity into realistic acceptance criteria.

NDE OF CONCRETE BRIDGES: OPPORTUNITIES AND RESEARCH NEEDS

Nicholas P. Jones[1] and Bruce R. Ellingwood[2]

ABSTRACT

The massive investment made in the nation's highway infrastructure over the past several decades is at risk from deterioration compounded by inadequate inspection and maintenance procedures. In the last several years, however, this problem has been recognized and concerted efforts are being made on the part of local, state and federal agencies to address it. One of the most critical needs is the development of appropriate nondestructive evaluation technology to facilitate the assessment and rational prioritization of (generally overdue) maintenance actions in light of the often restricted budgets available for such activity.

A large proportion of the bridges in this country are constructed (in whole or in part) from concrete -- reinforced, prestressed, or a combination of both. This material, by its very composition and potential degradation characteristics, poses unique challenges for bridge engineers and potential nondestructive evaluation techniques. The expected return for the development of appropriate techniques, however, is significant.

This paper addresses the problem of the deterioration of concrete bridges. However, much of the discussion is generalizable to other bridge structures (e.g., the concrete decks of composite structures) and other structures in general (e.g., concrete pavements, parking garages, etc.) Consistent with the goals of the workshop and composition of the audience, the paper is designed to provide a foundation for later discussion by specifically outlining the nature of the problem and the context in which it should be viewed. The paper begins with an overview of the problem facing the nation's highway infrastructure, followed by an

[1]Assoc. Prof., Dept. Civ. Engrg., The Johns Hopkins Univ., Baltimore, MD 21218-1686

[2]Prof. & Chair, Dept. Civ. Engrg., The Johns Hopkins Univ., Baltimore MD 21218-2686

overview of concrete bridge structures: their components, classification, existing design methodologies, inspection criteria, etc. A summary of the types of deterioration is presented, coupled, as appropriate, to descriptions of the characteristics of the reinforced/prestressed concrete composite material. The significance of the application of NDE techniques to structure assessment and longevity will be discussed; i.e., how will the data be used, and what are the likely improvements in performance that may result. An outline of applicable NDE techniques that are appropriate for structures of this type -- while beyond the scope of this paper -- is given in summary form. The reader is also referred to the paper by Allen (appearing in these proceedings) for more discussion of specific examples of deterioration and of suitable (or potentially suitable) NDE techniques.

1. INTRODUCTION

1.1 BACKGROUND

Despite relatively heavy investment in the US transportation infrastructure over the past several decades, which has included the design and construction of an extensive interstate highway system, the disturbing fact remains that a large proportion of this nation's bridges are in comparatively poor structural condition. The Federal Highway Administration has estimated[3] that of the approximately 578,000 bridges in this country, 136,000 are considered structurally deficient and another 103,000 are functionally obsolete[4]. Those 136,000 bridges -- 24\% of the total in service --represent a significant economic investment and pose a threat to life and limb should they fail catastrophically. The scope of the problem is broad, affecting all fifty states and the District of Columbia (Table 1).

[3]A word of caution is in order with regard to these statistics. Due to the perceived inadequacy and punitive nature of the extant rating procedures in many cases, it is believed that these statistics may overestimate the some of the numbers (Tarricone 1990); i.e., many bridges in fact have capacities exceeding those predicted by the current procedures. This does not, however, completely invalidate the numbers cited or the seriousness of the problem.

[4]"Functionally obsolete" means that the structure cannot be rated under existing procedures for the current load requirements due to inadequate or outdated initial design procedures, increased ratings (volume, loads) of adjacent highways, or deterioration. The structure will still accommodate restricted vehicular traffic. Structurally deficient means that the structure is effectively in an unsafe condition under normal service loadings.

The importance of this problem has recently been recognized by the Office of Technology Assessment of the U.S. Congress, which has recently organized a study entitled *Infrastructure Technologies: Rebuilding the Foundations*. The annual Transportation Research Board meetings now report much effort in this field. The American Society of Civil Engineers (ASCE) and the National Science Foundation (NSF) have recently sponsored a number of workshops on this topic, the most recent being the Civil Engineering Research Foundation Workshop in 1991. The document reporting the recommendations of this Workshop (ASCE 1991a) gives high priority to the repair and rebuilding of the transportation infrastructure. As noted in Weil (1989), the National Research Council in its report *Infrastructure for the 21st Century* recommends the development of user-friendly NDE systems, increased training, standardization of methods and formation of the central organization that could accelerate the use of NDE techniques for public works.

The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 requires issuance by December 18, 1992 of regulations for State development, establishment and implementation of systems for managing (1) Highway pavements, (2) *BRIDGES*, (3) highway safety, (4) traffic congestion, and (5) public transportation facilities, and (6) intermodal transportation facilities and systems. (Federal Register, Vol 57, No. 107, Juné 3, 1992). States are to begin implementing these management systems in FY 1995. The technology required to develop and maintain these management systems must be in place during the next three years. Currently, regulations are being prepared to implement the provisions of ISTEA.

Bridge Management Systems (BMS) are in a more advanced state of development than some of the others above. The American Association of State Highway and Transportation Officials (AASHTO), through the National Cooperative Highway Research Program (NCHRP), has produced a guideline on BMS that recently has been published. Some of the questions raised include:

1. Who has the responsibility for data collection?
2. To what extent is the standardization in data collection procedures necessary?

3. How can the conflict between the current system of reporting conditions in National Bridge Inventory vs. the detailed descriptions required in the proposed BMS, which requires more detailed information on deterioration, be resolved?

Perhaps one of the significant weak links in the BMS chain is the so-called "data:" What are they, how will they be collected and organized in a rational manner, and how will they be used? Criticism has been leveled at the potential use of life-cycle cost criteria in evaluating alternative structural materials and designs for similar reasons. The data to support such operating models are simply not available (ASCE 1991b). Some of these data include inspection and maintenance information. NDE approaches potentially provide solutions to this problem of lack of quantitative data. If the information (generally at the small or local scale) can be effectively incorporated into a rational structural condition and reliability assessment program, the long- and short-term benefits are potentially great.

Detection of deficiencies in bridges is a difficult and demanding task. While technologically sophisticated damage detection systems have been under development for a number of years, most bridges are still inspected by visual means. Although this method is satisfactory for the detection of gross flaws in bridge structural components, the likelihood of finding small or hidden defects is very low.

In addition to the identification of structural deficiency (with the associated safety implications) it is noted that significant economic gains are to be made with more accurate ratings: Lifting of load restrictions; extension of service lives of existing bridges; and postponement of costly (or even unnecessary) maintenance or strengthening projects.

1.2 GOALS

In the last decade, the infrastructure deterioration problem has been recognized and concerted efforts are being made on the part of local, state and federal agencies to address it. One of the most critical needs remains the development of appropriate nondestructive evaluation technology to facilitate the assessment and rational prioritization of (generally overdue) maintenance actions in light of the often restricted budgets available for such activity.

In the context of this workshop, the goals must be -- based on an identification and delineation of critical degradation mechanisms that affect reinforced and prestressed concrete structures -- to
review and assess existing NDE technology and develop recommendations for specific avenues of research which will provide the data necessary for the development of comprehensive bridge management systems. Existing assessment procedures are labor intensive, inconvenient and often unreliable. New technologies (e.g., radar, infrared thermography, laser optics, ultrasound, etc.) used independently or in parallel potentially offer the possibility of much more reliable and expedited inspections (Maser 1989).

Some of the fundamental technology already exists. The challenge will be in the adaptation of this technology to the bridge inspection problem. In other cases, new or emerging technologies will be required. Using the workshop as a vehicle, promising technologies -- along with new or unresolved research issues -- should be identified and prioritized for funding.

2. CONCRETE BRIDGE STRUCTURES

Concrete is one of the most widely used materials in modern bridge construction. Concrete is a durable material when properly placed, cured and maintained. However, there are a number of factors that can compromise its performance in service and impact its structural properties over time in a negative way. These factors include faulty design, unsuitable materials, improper workmanship, exposure to aggressive environments, excessive structural loads, and accidental loading conditions and abuse. Concrete is subject to a phenomenon known as aging, in which the material and structural properties change over time. Some of these aging effects are relatively benign, and in fact have a positive effect on strength. Others, however, may cause the load-carrying capacity to decrease and may increase the risk to public safety if not properly controlled. In-service inspection and maintenance provides a means for minimizing the impact of aging on performance. Nondestructive evaluation (NDE) methods and, in particular, those that are non-invasive and do not interrupt the use of the bridge, are essential for a properly designed in-service inspection and maintenance program.

In the reinforced-concrete composite, the low tensile strength concrete contains a number of steel reinforcing bars in the tension zones of the structural element. They are also often found in compression zones. Resistance of the structural member to bending, shear and axial loads is effected through this "sharing" of the stress components: The concrete takes compression and the steel (in general) resists the tensile stresses.

Under normal loading conditions, the chemical adhesion between the cement and the rebars would be overcome, and the tensile stresses would be transferred to the steel through friction and wedging action of dislodged particles. The efficiency of the stress transfer is enhanced by using deformed bars. It is noted that -- by design -- the concrete in the tension zone *is assumed to crack* under ultimate load conditions to effectively transmit the tensile stresses to the steel rebar. Control of this cracking is possible, and usually implemented to avoid the occurrence of wide, gaping cracks; however, cracks will be present in general, potentially exposing the reinforcement to corrosive agents.

Prestressed concrete is fundamentally different in behavior. By pretensioning the steel rods or tendons, the entire concrete component is placed in an initial compressive state. Bending moments are resisted by an internal couple resulting from reduced and increased compressive stresses. Properly designed prestressed concrete does not crack under service loading conditions. However, due to the fabrication or construction techniques used, the potential for corrosive agent infiltration still exists; the critical dependence of the structure on the tension in the prestressing tendons still make corrosion a particular concern.

2.1 OVERVIEW OF BRIDGE STRUCTURES

It is appropriate to begin this discussion with a clear definition of the terminology relevant to concrete (and most other) bridge structures. The basic components of a bridge are classified as follows (see Figure 1):

- The foundation refers to the ground on which the piers are supported. A more general definition also would include the foundation "mat" -- generally a thick reinforced-concrete slab -- or the piling system -- usually long steel or prestressed concrete members -- designed to distribute the relatively concentrated pier load to the surrounding soil[5].

- The abutment is a special foundation which is associated with the ends of the bridge structure. It almost always includes the approach roadway, and in many cases is built up above the surrounding ground level. The connection between the bridge superstructure and the abutment is usually specially designed to not only support the vertical dead (generally weight) and live (generally traffic, including impact effects) load, but also to accommodate differential axial motions accompanying the expansion and contraction of the structure due primarily to thermal effects.

- Piers are vertical column-type components which carry the predominantly vertical load from intermediate bridge supports to the foundation substructure.

[5]There is one special category of deterioration or failure which potentially afflicts a large number of bridge structures: foundation settlement (or differential settlement) wherein the soil on which the superstructure is founded loses bearing capacity. One particularly important case of this phenomenon occurs for river-crossing bridges. The flow of water past the foundation "scours" the foundation soil (generally sand and gravel) away from the foundation, often eventually causing loss of foundation capacity and even failure of the entire superstructure. A number of notable recent bridge failures in recent years (which have led to fatality) have been scour-induced; for example, the Scoharie River Bridge on the New York Thruway. While beyond the scope of this paper, it is clear that development of suitable NDE techniques to identify scour conditions is essential for all bridges for not only maintenance but also life safety considerations.

- Girders are generally the main load-carrying structural members in the superstructure, conveying the spatially distributed dead and live loads from the deck to the piers and abutments. These members are generally standard or built-up steel sections, or reinforced- or prestressed-concrete members. Being primary members, their structural integrity is of prime concern.

- The deck is the *structural* component which is designed primarily to distribute the spatially distributed vehicle loads to the remainder of the superstructure. This subsystem is usually designed as a concrete (reinforced, prestressed, or both) slab, although steel is also used. Indeed, the deck is a critical element in the present discussion for both concrete and composite construction. In the U.S., data suggest that while bridges last on average 68 years, the decks only last 35 years before needing major repair or replacement (Bettigole 1990). Indeed, of the functionally obsolete or structurally deficient bridges mentioned earlier (approximately 42% of the total), two-thirds were in that category because of failed concrete decks (Wolchuk 1988).

- The wearing surface is the slab overlay which provides the usable surface of the bridge for traffic. In many cases, the deck surface itself fulfills this function, but often there is a concrete or asphalt overlay over the deck structure. This surface -- although usually considered to be sacrificial and often designed to be replaced -- is critical for a number of reasons: for example, it provides the driving surface, and therefore must be maintained in a safe and usable condition; it protects the (structural) deck below from direct wear and the penetration of corrosive elements.

While these components are common to almost all types of structure, the classification of the bridge structure as steel, concrete, composite, etc., depends on the materials used to fabricate the components. For example, a "concrete bridge" may have a reinforced concrete footing and piers, reinforced- or prestressed-concrete girders, a reinforced- or prestressed-concrete deck, and an asphalt wearing surface. A composite structure usually refers to the girder/deck combination: generally, the steel girders are structurally connected to the concrete slab in a manner to ensure the two elements act together as a single structural element. Many of the comments in this paper referring to concrete decks are indeed applicable to the decks of both concrete and composite structures.

A number of different basic configurations are used in bridge construction, ranging from very simple arrangements for short spans to highly complex prestressed or cable-supported structures. Typical bridge structural forms are outlined below (Winter and Nilson 1979). In most cases -- unless specifically noted -- "concrete" refers to reinforced concrete, prestressed concrete, or a combination of the two.

- Simple, one-way slab construction (Figure 2a) is often used for short spans (10 to 25 feet) For structures of these dimensions, the internal loads induced in the structure are sufficiently small that the deck slab itself is the only structural member necessary. For slightly longer spans, hollowed sections may be used to achieve a higher strength-to-weight ratio (Figure 2b). These units are often precast, enabling simple and fast erection procedures.

- Precast, integral deck units (Figure 3) are also popular for short-span structures. These factory-produced systems offer the same economy and ease-of-construction advantages as their counterparts above. Prestressing -- both pretensioning (during fabrication) and post-tensioning (on site) -- is most commonly used.

- For spans up to 100 ft., cast-in-place girders (Figure 4) can be used. The integral nature of the deck with the girders provides structural efficiency. These may be designed as simply supported, single-span structures, or in a continuous, multi-span configuration to take advantage of the favorable distribution of internal forces that results.

- Composite steel (Figure 5) or prestressed (Figure 6) girder-concrete deck construction is the most common for medium-span (60 to 100 feet) structures in the U.S. The girders are of steel or prestressed concrete, and the deck is poured-in-place concrete. Shear connectors are provided at the top of the girders which lock these members to the deck to enhance the structural efficiency. Again, advantages of speed and simplicity of construction are evident.

- Hollow box girders (usually prestressed) are used for intermediate to long spans. Sections up to 80 ft. in length can be prefabricated and lifted into place. For longer spans, segmental box-girder construction has become popular wherein the balanced cantilever method is used to progressively erect the structure by lifting sections with cranes and then anchoring them in place by post-tensioning to the existing structure.

- Concrete arch construction (made popular by the aesthetically -- and structurally -- pleasing designs of Maillart earlier in the century) is not particularly common due generally to the special-purpose nature of the designs.

- Cable-supported structures are necessary for long, high spans e.g., over navigable waterways. These structures are of the classical suspension of cable-stayed variety. Newer structures are more commonly of the latter form, due to the economies of design. The main deck structure may be of steel, concrete or composite construction. Relevant examples are the Sunshine Skyway Bridge in Florida (with a prestressed-concrete, box-girder system) and the new Houston Ship Tunnel (Baytown) Bridge under construction in Texas (composite deck). Detailed discussion of the deterioration problems associated with these structures is beyond the scope of the paper, but some concerns have been recently raised regarding corrosion of the cable systems (e.g., Watson and Stafford 1988).

2.2 CURRENT DESIGN REQUIREMENTS FOR BRIDGES

Most bridges in the U.S. are designed in accordance with the *AASHTO Standard Specifications for Highway Bridges* (AASHTO 1989). These comprehensive provisions provide recommended design loads, analysis and design procedures for various material types, material specifications and requirements, construction specifications, etc. While it is beyond the scope of this paper to outline in detail analysis and design provisions, it is prudent to highlight some of the features of the current practice and procedures.

2.2.1 Loads

Bridges must be designed to resist a variety of loads and load combinations. These include dead load (weight), live load (generally the design truck load), impact, wind, earthquake, thermal stresses,
earth pressure, etc. For brevity, this paper will discuss the first two categories only. Dead loads are relatively straightforward to estimate. While for buildings, determination of appropriate live load conditions is also reasonably well defined, for bridges this presents a challenge. While the "standard trucks" can be defined as a series of wheel loads, the spatial and temporal distributions of these trucks over the span and lanes requires careful (and statistical) treatment. Design loads are specified as the sum of a concentrated load plus a uniform load per foot of lane. Loads are combined in an appropriate manner for the design approach.

2.2.2 Design

Existing versions of the code permit the use of the *Service Load Design* method (also called the allowable stress design method) or the *Strength Design* method (also called load factor design method). In the former, loads are combined by simple addition (in general) and the resulting stresses are computed elastically; these stresses must be less than the yield or other limiting stress reduced by what is essentially a safety factor. In the strength method, loads are factored before combination, and the resulting member forces are checked against the ultimate strength (multiplied by a capacity reduction factor which varies with the function of the member.)

A new version of the *Provisions*, currently in development, presents another method called the *Load and Resistance Factor Design Method* (LRFD). LRFD has been incorporated into building codes for some time, but is only now being considered for bridges. The approach is based on a more rigorous probabilistic treatment of the uncertainties inherent in both the design loadings and the capability of the structure to resist them. It represents a significant enhancement to the Strength Method outlined above.

2.2.3 Crack Control and Corrosion Resistance

Because of the nature of reinforced concrete, consideration must be given to reduce the exposure of the system to corrosive action. In the *Provisions*, this is effected through

- Specification of minimum steel requirements to ensure well-distributed small cracks due to shrinkage and thermal expansion rather than isolated large cracks, and

- Specification of minimum cover between the outer surface of the member and the reinforcement concrete to reduce the exposure of the rebar to air, water, and corrosive agents.

The former is not of major concern for prestressed concrete due to the manner in which the member

resists load (i.e., it remains in compression and does not crack.) However, shrinkage and creep can affect the level of prestress force (in addition to friction losses and elastic extension/compression), and therefore must be considered for this reason.

2.2.4 Maintenance

Bridge engineering must cover not only the design and construction of bridges, but also their operation, inspection and maintenance. In this context, AASHTO also publishes the *Manual for Maintenance Inspection of Bridges* which outlines recommended procedures relative to personnel qualification, responsibilities, suggested frequencies, etc.

Periodic inspection of bridges is required for a number of reasons:

- protection of investment
- safety
- bridge rating
- permitting of overweight vehicles
- limiting vehicle weights of deficient structures

A registered engineer should be in charge of the inspection process, and has the responsibility for both directing the physical inspection and analyzing and assimilating the results. This person must be not only familiar with bridge design and construction, but also familiar with (or have access to appropriate personnel who are) the many techniques used and able to evaluate capacity and recognize deficiencies and their seriousness.

AASHTO recommends that bridges be inspected at regular intervals, not to exceed two years. In some special cases, the inspection procedure should be carried out with much shorter intervening periods. They suggest that States inventory their structures and evaluate the appropriate frequency and techniques to be used for each bridge on a case-by-case basis. In practice, due to omnipresent budget constraints, planned periodic inspection is not conducted, and inspections only performed when problems are identified.

While new and sophisticated techniques are continuously being proposed, developed and implemented for bridge inspection purposes, these are often highly specialized and appropriate only for the specific task for which they were developed. By far the most common (and comprehensive) bridge inspection procedure is visual inspection by a trained team, which records the condition of the structure -- usually concentrating on potential "problem areas" identified in advance. Based on the data collected, calculations are performed, if necessary to quantitatively evaluate the structural condition, and recommendations made for action. More detailed procedures are sometimes requested.

3. SOURCES AND MECHANISMS OF DEGRADATION OF CONCRETE BRIDGES

Reinforced concrete degrades in predictable ways during its service life (Somerville 1986). The surface can wear and spall from the impact of vehicular traffic. This type of damage is visible at the surface, and can be controlled with a program of regular maintenance. Less obvious is the accumulated damage to the reinforcing bars from environmental factors. Freshly cast concrete is highly alkaline, and in this state protects the steel from corrosive action. With time, the concrete can carbonize and lose its passivating action. Or, corrosive chemicals such as de-icing chlorides can penetrate -- by diffusion, percolation, or ingress along crack faces -- to the steel-concrete interface. In the presence of an electrolyte, corrosion proceeds rapidly. Products of the electrochemical reaction tend to build and cause large hydrostatic pressures to accumulate at the interface. This pressure results in tensile forces in the concrete which cause cracking, spalling and local failure of the composite material.

Rather than present case studies of bridges, it was considered more appropriate for the context of the workshop to give a description of environmental stressors and mechanisms of attack that may lead to deterioration of concrete bridge structures. Understanding the mechanism by which degradation occurs is important in determining appropriate NDE techniques to measure or assess it. The discussion addresses both concrete and steel separately, although there is clearly some important commonalities in these mechanisms.

One disadvantage of presenting the material in this way is that some of the unique characteristics of bridge structures relative to the applicability of NDE techniques are omitted. Because of the morphology of these structures not all *feasible* NDE techniques will be *practical*. This is an important caveat, and will no doubt be discussed more fully during the workshop.

In the sections following, degradation mechanisms are subdivided into chemical and physical attack for convenience. Figure 7 illustrates the time-dependent behavior of a selection of the important mechanisms.

3.1 CONCRETE MATERIAL SYSTEMS

3.1.1 Chemical Attack

Hydrated cement is an alkaline material. Alteration of concrete occurs through chemical reactions with either cement paste or the aggregate. The effect is most pronounced on the surface of the concrete, but may affect the entire structure if cracking allows the ingress of aggressive substances. The rate of chemical attack depends on the concrete permeability and reactivity and the pH of the aggressive substance. Expansive reactions cause cracking, accelerate the penetration of aggressive substances, and may lead to strength deterioration.

<u>Sulfate attack</u> is a common problem for concrete in contact with soils (Lauer 1990), particularly in portions of the western United States where naturally-occurring sulfates of calcium, sodium, potassium, and magnesium are found in soil and groundwater. Degradation from sulfate attack is accelerated by wetting and drying cycles (the area of the foundation near ground level is particularly susceptible) and occurs by a combination of convection and absorption (Clifton 1991). Sulfate attack results in expansive internal reactions, manifested by pattern cracking and subsequent disintegration of concrete when the expansion reaches about 0.5 percent. The depth of sulfate attack appears to be linearly dependent on time (Harrison and Teychenne 1981; Walton, et al. 1990) i.e.,

$$x(t) = K \cdot t \tag{1}$$

in which $x(t)$ = depth of deterioration, t = elapsed time, and K = constant dependent on the amount of tricalcium aluminate in the concrete and the concentration of the sulfate environment. A nonlinear attack relation proportional to t^n with $n>1$ has also been suggested to model loss of section (Clifton and Knab 1989).

Alkali-aggregate reactions arise from reactive aggregates in the concrete mix and thus, unlike most of the other mechanisms considered, is not dependent on an aggressive external factor. Expansive alkali-aggregate reactions require a source of water. Such reactions can occur mainly with siliceous (alkali-silicate reactions: ASR) (and, infrequently, with carbonate) aggregates over wide areas of the United States. The expansive reaction may lead to the development of internal stresses and cracking of the concrete. Such cracking tends to be uniformly distributed over a wide area of the surface of the concrete component, may increase its vulnerability to other aggressive factors, and can lead to reinforcement corrosion.

Degradation in compressive and tensile strength and in modulus of elasticity due to ASR depends on the expansion (Clark 1990). The degradation appears to be approximately a linear function of expansion; e.g., at 5 mm/m expansion, f_c decreases to about 0.85 f'_c, f_t decreases by about 30%, and E decreases by about 50% (Clark 1988; 1990). These decreases were obtained from tests of small unrestrained specimens. The effect of ASR in structures in which the concrete is properly restrained by reinforcement or compressive forces normally is relatively benign in comparison to the behavior inferred from small laboratory specimens.

ASR damage normally becomes apparent within a few years of construction; however, there have been cases where damage did not become apparent until 20 years after construction or where the reaction continued after 40 years or more (Clark 1988; Clifton and Knab 1989; Berra and Bertacchi 1991). Concrete expansion of 0.08 mm/m/yr has been measured in large dams (Berra and Bertacchi 1991). No mathematical model to predict structural deterioration due to alkali-aggregate reaction seems to be available, although there is evidence that the basic reaction is diffusion-controlled (Clifton 1991).

Leaching of calcium hydroxide causes a reduction in pH and in concrete strength. Concrete degrades in strength as soluble constituents of the concrete are dissolved in the presence of water. If the nonsoluble products recrystallize and expand in the pore system, tensile stresses occur that may lead to surface cracking. The decline in pH also may initiate corrosion of reinforcement. The rate of leaching is dependent on the permeability and porosity of the concrete (Clifton and Knab 1989). The process is diffusion-controlled and thus the decrease in strength is approximately proportional to the square root of time (Shuman, et al. 1988).

Creep refers to accumulated deformation of concrete under stress and can cause cracking of mass concrete subjected to change in temperature and increased deflection (Sansalone and Carino 1991). Volume changes due to shrinkage are caused mainly by drying and continued hydration. Creep and shrinkage are the main sources of prestress loss in prestressed concrete structures. Neither creep nor shrinkage affect the load-carrying capacity of the structure at near-ultimate conditions however, creep and shrinkage may be important for serviceability limit states.

Acid attack. As an alkaline material, concrete is vulnerable to acid attack. Sulfuric or carbonic acids in groundwater are among the sources of sources of acid attack that may occur on concrete structures at or below grade. The chemical reaction of acidic solutions with the constituents of portland cement paste gives rise to soluble salts of calcium that leach out and increase the permeability of the concrete.

Salt crystallization. When concrete is in contact with water containing dissolved salts and water permeates the concrete, salts may crystallize in pores within the concrete from evaporation. Repeated evaporation can cause salt deposits to build up, leading to large tensile stresses and cracking of the concrete. Concrete structures subjected to fluctuating water levels are particularly susceptible to this damage mechanism.

3.1.2 Physical Attack Mechanisms

Freeze-thaw cycling is mainly a problem when exposed damp concrete is subjected to cycles of freezing and thawing. Horizontal concrete surfaces such as pavements, bridge decks, roofs, sills and other surfaces where water can remain in contact with the concrete for sometime are especially susceptible to freeze-thaw damage (Clifton and Knab 1989). The critical level of saturation for freeze-thaw damage to occur is about 85%. Water when it freezes increases in volume by about 9 percent, leading to cracking and spalling (Bryant and Mlakar 1989).

Elevated temperatures cause decreases in compressive strength, tensile strength, and modulus of elasticity of concrete. However, tests have shown that significant degradation does not occur until temperatures reach 200 C or more. Such temperatures are not reached in a bridge deck. Although thermal cycling at lesser temperatures may cause some degradation, the presence of even nominal amounts of reinforcement usually prevents damage from occurring. Accordingly, degradation of strength or stiffness due to elevated temperature effects can be neglected.

Fatigue. Fluctuations in loading or temperature can initiate fatigue damage in the form of microcracks in the cement paste, at cement-aggregate interfaces, or in the reinforcement. Under reversals of stress, these microcracks may propagate to form structurally significant cracks that increase member flexibility and admit aggressive substances, enhancing further deterioration. The fatigue strength of a properly designed and constructed concrete member at a life of 10^7 cycles is approximately 55% of its static strength. Bridge structures are known to be susceptible to fatigue damage accumulation, and a bridge management program should take this mode of degradation into account.

Abrasion leads to progressive loss of material at the concrete surface. Resistance of concrete to abrasion depends on the porosity and strength of the concrete and the quality of the aggregate in the mix.

3.2 STEEL REINFORCEMENT

Perhaps one of the most insidious forms of accumulated damage to bridges is corrosion (ACI 1985). Due to the nature of the environment to which many bridge structures are subjected, excessive corrosion with resultant strength degradation is a commonly encountered problem. The steel tendons found in concrete structures, either in reinforced or prestressed material, are particularly susceptible to corrosive action because of the very nature of the composite, its fabrication and morphology. The presence of corrosion can -- and has -- led to failures of concrete structural members. Mechanically, there is inadequate shear stress transfer between the concrete and the corroded rebar, severely degrading the tensile strength of the composite. Timely identification and location of this condition, usually concealed in the concrete, would permit suitable remedial action to be taken, thereby averting a possible disaster.

Corrosion of prestressing tendons has been found to occur near voids in the grout used to fill post-tensioning ducts, and in the cast-in-place steel reinforcement (Tilly 1990). These conditions have led to sudden, unanticipated failure of bridge structures. While careful attention is given to crack control in reinforced concrete, occurrence of cracks and microcracks is a feature of the composite material. These and diffusion allow environmental influences, principally chlorides from de-icing salts, to reach and corrode the reinforcing steel, potentially weakening the entire structure.

Although corrosion can be a problem with both deformed bar reinforcement and prestressing tendons, it is less likely to be a problem for tendons because they usually are contained in ducts which are packed with grout or corrosion-inhibiting grease under pressure. In some structures, such as bridges, the tendons are inspected periodically for corrosion. Sections of heavily reinforced components where the concrete is difficult to compact properly may also be vulnerable to corrosion (Sentler 1983).

Protection of reinforcement against corrosion involves a physical barrier and a chemical barrier. The physical barrier is related to the amount of concrete cover and the presence of cracks. The chemical barrier comes from the alkalinity of portland cement concrete, which passivates the reinforcement surface and provides a natural barrier against corrosion (Neville 1983). Both water and oxygen must be present for corrosion to occur. The presence of chlorides in combination with relatively high humidity in the concrete can cause the active corrosion rate to accelerate to the point where it can become a serious structural problem in only a few years. The corrosion products are several times in volume that of the uncorroded steel. Tensile forces that develop in the concrete from volume expansion of the corroding steel cause cracking and subsequent destruction of the concrete cover and a decrease in structural capacity dependent on the area of reinforcement lost due to corrosion. Corrosion products also can cause deterioration in the bond between reinforcement and concrete.

While corrosion of steel girder bridges is often detectable by visual means and measurement of exposed surfaces, determining the state of reinforcing bars or prestressing tendons within a concrete member is much more challenging. Existing methods for determining such corrosion are generally destructive, in that access holes must be bored in the member and an assessment of deterioration made directly. Not only is this procedure damaging to the structure, but also limited in spatial applicability, time-consuming, and expensive.

Carbonation. Penetration of the concrete by atmospheric carbon dioxide reduces the natural alkalinity of the concrete (Parrott 1990). When the penetration reaches the reinforcement, it destroys the passive oxide layer protecting the reinforcement and allows active corrosion to initiate. Carbonation is basically a diffusion-controlled process. The depth of carbonation, x(t), is given as,

$$x(t) = K \sqrt{t} \tag{2}$$

in which t = elapsed time and K = constant dependent on the composition of the concrete and the concentration of carbon dioxide in the environment.

Chloride Ion Attack. Chloride ions may be present as a result of deicing salts, natural constituents of certain aggregates, and certain chemical admixtures (Browne 1982). The rate of penetration appears to be diffusion-controlled. The diffusion of chloride ions to the depth of the reinforcement can be predicted by (Vesikari 1988; Walton, et al. 1990; Clifton, 1991),

$$\frac{C}{C_s} = 1 - erf\left(\frac{x}{2\sqrt{D.t}}\right) \tag{3}$$

in which C = chloride concentration for initiation of corrosion, C_S = surface chloride content, D = diffusion coefficient for chlorides, and erf (•) = error function. There is substantial uncertainty in the diffusion coefficient (Clifton 1991). Setting C equal to the critical concentration for initiation, this equation may be solved for the initiation time, t_I, required to depassivate the surface of the reinforcement.

Active corrosion begins once the level of carbonation or aggressive ion penetration has reached the reinforcement. Once active corrosion has initiated, the corrosion rate, r_c, is dependent on the corrosion current, area of anodes, relative humidity and temperature (Tuutti 1982). The depth of penetration of active corrosion, x(t), is given as,

$$x(t) = r_c \, . \, t \tag{4}$$

If the propagation time is evaluated as the time required for spalling of the concrete cover, the period of active corrosion, t_P, is given as,

$$t_P = k_a \, . \, c \tag{5}$$

in which c = reinforcement cover and k_a = constant. The total service life, t, would then be (Vesikari 1988),

$$\begin{aligned} t &= t_I + t_P \\ &= K \, . \, c^2 + k_a \, . \, c \end{aligned} \tag{6}$$

in which t_I = time required to initiate active corrosion. In most instances, t_P is substantially less than t_I. Moreover, the level of uncertainty in t_P is much higher than that in t_I.

It has been suggested (Browne 1982; Clifton 1991) that the service life of concrete should be taken as the time required for corrosion to initiate. This is particularly important for prestressing tendons where, because of their relatively small area, the initiation of active corrosion means that the end of their service life is imminent. The rate of penetration of chloride ions through concrete has been shown to be higher than other common degradation processes, such as leaching, sulfate attack and ASR. In one study to determine longevity of concrete waste repositories with a minimum desired service life of 500 years, the time required for chloride ions to penetrate to the level of the reinforcement was taken as a conservative criterion for failure (Philipose, et al. 1991). Similar proposals have been made for predicting durability of concrete structures in marine environments (Browne 1982).

Degradation of the post-tensioning system is a concern in prestressed concrete structures. In some structures such as bridges, periodic inspection of tendons is required. Hydrogen embrittlement or stress-corrosion cracking, crushing of concrete under the anchors, relaxation of the tendons, and microbiological corrosion of the tendons all are possible degradation mechanisms (Shah et al. 1988). Detensioning does not change the ultimate load-carrying capacity of a concrete component significantly; however, it does change its load-deformation characteristics and the load-resisting mechanisms at loads less than ultimate load, and increases the likelihood of crack formation, subsequent reinforcement corrosion, and leakage.

End effects. The process of post-tensioning tendons or retensioning in compliance with inspection programs can lead to localized damage in the vicinity of the anchor due to the large local concentrated forced introduced by this process. Tendon anchorages can be the sites of high stresses, local damage and occurrence of other damage mechanisms.

4. TIME-DEPENDENT STRUCTURAL PERFORMANCE OF BRIDGES

A bridge management system should provide a set of criteria, rules and quantitative tools that can be used to evaluate bridge structures and components for fitness for continued service. The evaluation of concrete bridge structures for continued service should provide quantitative evidence that their strength is sufficient to withstand future extreme events within the proposed service period with a level of reliability sufficient for public safety.

As noted previously, aging causes the strength and serviceability of concrete structures to evolve over time, and may impact the safety of a bridge structure. The aging phenomenon must be taken into account in the development of bridge management systems. There are numerous sources of uncertainty that complicate the evaluation of aging effects on the residual strength of a concrete bridge. Uncertainties arise from (1) differences in design codes and standards for bridge components of different ages; (2) lack of in-service measurements and records; (3) limitations in available models for quantifying time-dependent material changes and their contribution to bridge structural strength; (4) inadequacies in nondestructive evaluation (NDE) technologies; and (5) shortcomings in existing methods for rehabilitation and repair.

During the past several years, research has been underway to develop methodologies for performing condition assessment and evaluation of reinforced concrete structures in nuclear power plants and hazardous waste depositories (e.g., Naus 1986; Ellingwood and Mori 1991). The methodologies integrate information on design requirements, degradation and damage accumulation, environmental factors, and NDE technology into a decision tool that provides a quantitative measure of structural reliability and performance under projected future service conditions based on an assessment of a new or existing structure. This research has highlighted the need for quantitative modeling of strength degradation and the impact on NDE on in-service condition assessment. Experience gained with these concurrent research programs can be used to identify desirable characteristics of NDE technologies to be used in bridge inspection and to optimize inspection and maintenance strategies.

Structural loads, engineering material properties and strength degradation mechanisms are random in nature. Time-dependent reliability analysis methods can provide a framework for performing condition assessments of existing structures and for determining whether in-service inspection and maintenance is required to maintain reliability and performance at the desired regulatory level (CIB 1987).

It is of interest to illustrate some of these concepts using some recent developments in the area of aging concrete structures in nuclear power plants (Ellingwood and Mori 1991). Assume that significant structural loads (due, perhaps, to the passage of large trucks) can be modeled as a sequences of pulses, the occurrence of which is described by a Poisson process with mean rate of occurrence, λ, random intensity, S_j, and duration τ. This load process model can be modified, if necessary, to account for nonuniform load occurrence rate and intensity, but this will not be done in the simple illustration of the concept to follow. At the same time, the strength of the concrete structure decreases over time due to environmental attack. This situation is illustrated conceptually in Figure 8. The load statistics can be obtained with current bridge monitoring technology. The behavior of the resistance over time must be obtained from mathematical models describing the degradation mechanism(s) present, as described in Section 3. The statistics used in the illustrations to follow are available elsewhere (Ellingwood and Mori 1991) but are not presented as they do not contribute to the basic understanding of the concept.

The reliability function, L(t), is defined as the probability that the structure survives during interval of time (0,t). If n load events occur within time interval (0,t), at times t_1, t_2, t_n, the reliability function can be represented as (DeKraker et al. 1982),

$$L(t) = P[R(t_1) > S_1, \ldots, R(t_n) > S_n] \tag{7}$$

in which P[ ] is the probability of the event in brackets. Taking into account the randomness in the number of loads and times at which they occur, the reliability function becomes,

$$L(t) = \int_o^\infty \exp[-\lambda[t- \int_o^t F_s\ (r.g(t))\ dt]]\ f_R(r)dr \tag{8}$$

in which $f_R(r)$ = probability density function of initial strength, R and g(t) is a function describing the degradation in strength with time. The limit state probability, or probability of failure during interval (0,t) can be determined as,

$$F(t) = 1 - L(t) \tag{9}$$

The hazard function, h(t), is defined as the probability of failure within time interval (t,t+dt), given that the component has survived up to time, t. This conditional probability can be expressed as,

$$h(t) = - \frac{d\ln L(t)}{dt} \tag{10}$$

When structural failure occurs due to aging or deterioration, h(t) increases with time. In-service inspection and maintenance impacts the hazard function, causing it to change discontinuously at the time ISI is performed. We will see this in the subsequent illustration.

The effect of degradation in strength on the reliability and hazard functions for a concrete component are illustrated using several simple parametric representations of time-dependent strength summarized below:

Degradation	Form	Typical mechanism
Linear	$g(t) = 1 - at$	Corrosion
Parabolic	$g(t) = 1 - at^2$	Sulfate attach
Square root	$g(t) = 1 - a\sqrt{t}$	Diffusion-controlled

Parameter a is simply an experimentally determined constant. Each reliability analysis was performed for a service life of 60 years. The degradation is defined with reference to a residual strength at 40 years, e.g., $g(40) = 0.8$ means that 80 percent of the initial strength remains at 40 years. Components analyzed were designed using the requirements of ACI 318 for flexure:

$$0.9R_n = 1.4D_n + 1.7L_n \tag{11}$$

in which R_n = nominal or code resistance and D_n, L_n = code-specified dead and live loads.

The effect of the general characteristics of the degradation function on F(t) and h(t) are presented in Figures 9 and 10. Up to 40 years, the failure probability associated with the diffusion model is the highest. However, after 40 years, the failure probability associated with the other models increases more rapidly. Note from Figure 9 the grossly unconservative effect of neglecting strength degradation entirely in a time-dependent reliability assessment. The increase in h(t) in Figure 10 is characteristic for aging structures; for random or purely chance failures, h(t) is constant.

Periodic in-service inspection followed by suitable maintenance may restore a degraded concrete bridge to near-original condition. Such inspection and maintenance strategies should be designed so that the failure probability of the component is kept lower than an established target probability, P_f, during the service life. Since inspection and maintenance are costly, there are tradeoffs between the extent and accuracy of inspection, required reliability, and cost. An optimum inspection and maintenance program might be obtained from the following constrained optimization problem:

$$Minimize\ C_T = C_{ins} + C_{rep} + C_f P_f \tag{12}$$

$$Subject\ to\ F(t) < P_f \tag{13}$$

in which C_T = the total cost, discounted to present worth, C_{ins} = cost of inspection, C_{rep} = cost of repair/maintenance, and C_f = cost of failure, including cost of social disruption due to failure of the bridge.

It is clear -- in light of the earlier discussion -- that the application of *quantitative* NDE techniques can affect all the terms on the right-hand side of the above equation. Cost C_{ins} depends on the NDE method(s) selected for the inspection. Existing techniques are labor intensive and rather costly as a result. While newly developed NDE methods and systems are likely to also be expensive, it is anticipated that these costs will reduce as they become more accepted and commonplace. The cost C_{rep} will also be affected by reliable NDE, although whether it will be increased or decreased is uncertain. With more dependable and accurate methods, flaws will be detected more reliably thereby reducing the uncertainty associated with them. This may lead to the discovery of heretofore undetected defects, which potentially increases the cost, but the more reliable assessment will hopefully enable the postponement of unnecessary repairs. This enhanced ability to differentiate between necessary and unnecessary repairs is expected, however, to improve the overall cost efficiency of the inspection and maintenance program. The dividing line between the flaws that the NDE method is able to or fails to detect will have a direct impact on P_f.

Time-dependent reliability analysis can be used to perform this minimum cost analysis. To illustrate this with a simple example, consider two alternative strategies: (1) infrequent but thorough inspection and maintenance performed at 20 and 40 years, with restoration of full strength, and (2) frequent but limited inspection and maintenance performed at intervals of 10 years, with restoration of 97% initial strength. The failure probabilities associated with these strategies (as well as with doing nothing) are compared in Figure 11. At the time of inspection, h(t) changes discontinuously and F(t) changes slope. If $P_f = 0.00025$ in 40 years, strategy (1) would be unacceptable, while strategy (2) would be acceptable. In this case, at least, frequent cursory inspection seems preferable to infrequent thorough inspection.

5. THE ROLE OF IN-SERVICE INSPECTION

5.1 IMPLICATIONS FOR STRUCTURE PERFORMANCE

Forecasts of reliability of the type illustrated in Section 4 above enable the analyst to determine the time period beyond which the desired reliability of the bridge cannot be ensured. At such a time, the structure should be inspected. Intervals of inspection and maintenance that may be required as a condition for continued operation can be determined from the time-dependent reliability analysis. A conceptual illustration of the effect of this process on the hazard function, h(t), is presented in Figure 12. That figure shows that the effect of inspection/repair is to remove larger defects from the structure and to upgrade its strength, thus reducing its conditional failure rate. As the structure ages, the failure rate increases until another inspection/repair operation occurs. The integrated effect of h(t) in Figure 12 must remain below the target limit state probability, P_f. In-service inspection and maintenance are a routine part of managing aging and deterioration in many engineered facilities, and work has initiated to develop policies for offshore platforms using probabilistic methods (Madsen, et al. 1989).

The effect of in-service inspection and maintenance on the frequency distribution of strength may be visualized in Figure 13. The frequency distribution of strength, based on prior knowledge of the materials in the structure, construction and standard methods of analysis is indicated by the curve $f_R(r)$. In-service inspection reveals additional information about the actual strength. Scheduled maintenance and repair also may cause the characteristics of the strength to change. The effect of inspection (and maintenance) may be visualized by the (conditional) frequency distribution $f_R(r|I,M)$, also shown in Figure 13. The time-dependent reliability analysis then is re-initialized following in-service inspection/repair using $f_R(r|I,M)$ in place of $f_R(r)$.

From the point of view of structural condition assessment, there are basically two categories of in-service structural evaluation methods. The first category involves NDE methods that detect the presence of defects in a structure. These include ultrasonic, pulse echo, magnetic, radiographic and electrical resistivity methods. Some are qualitative in nature, indicating the presence of a defect but not providing quantitative data on its size, precise location, and other characteristics that are required to determine the impact of the defect on structural performance. Such NDE methods are not especially helpful in structural condition assessment. Other methods provide quantitative data on defect size and location; these methods are of particular interest in support of condition assessment and reliability analysis. The second category includes methods that indicate the strength, directly or indirectly; these include drilled cores, penetration, rebound, or pull-out tests, or pulse velocity measurements (Nasser and Al-Manaseer 1987). Quantitative data on the capabilities of in-service NDE methods are required to determine appropriate modifications to the frequency distribution $f_R(r)$ in Figure 13.

In conducting a NDE, one might expect to find no defects, defects that do not present a safety problem under any conceivable circumstances, or defects that may grow to a dangerous size before the next scheduled inspection. It must be recognized that no NDE method can detect a given defect with certainty. This is particularly difficult with a heterogenous material like reinforced concrete. The imperfect nature of NDE methods must be described in statistical terms. This randomness affects the frequency distribution, $f_R(r\,|\,I,M)$, and the calculated reliability of the component. Figure 14 illustrates conceptually the probability, $d(x)$, of detecting a defect of size x. Such a relation exists, at least conceptually, for each in-service inspection method. It is important to identify detection probability curves for each NDE technology that might be used in a bridge management system. Moreover, it is important that the NDE method be able to distinguish between different defects with a high level of reliability so that needless repairs can be avoided.

Methods of testing for fitness-for-service must be tailored to the specific application at hand. No single technique or specific realization of a general approach will suffice for the inspection of every element in a structure as complicated as a bridge. When, as in the case of infrastructure inspection and qualification, there are many potential problems to consider and limited resources to attack them, the most effective approach often is to select an inspection of a safety-critical element in the structure and concentrate attention on that aspect. The strength of concrete decks and beams rests on the integrity of the steel reinforcing bars or prestressing tendons. Because of their criticality and difficulties in observing nucleation of corrosion, NDE methods for evaluating corrosion should receive particular emphasis.

5.2 AVAILABLE TECHNIQUES

The fundamental goals of a field investigation are to determine the extent of deterioration, establish as-built details, existing loadings, and exposure conditions, or a combination thereof (Pinjarkar 1990; Nowak 1990). Generally a program may include a series of physical measurements (concrete cover, metal loss, crack widths, crack patterns, deflections, movements at joints and bearings, etc.) and nondestructive evaluations using a variety of techniques (ultrasound, corrosion potential, infrared thermography/spectroscopy, ground-penetrating radar, acoustic emission, radiographic testing [x-ray diffractometry/florescence], differential thermal analysis, petrographic microscopy, etc.) (Pinjarkar 1990; Hime 1990; Clifton et al. 1982; Collacott 1985). The reader is also referred to Malhotra and Carino (1991) which provides an excellent overview of techniques for nondestructive evaluation of concrete and to Suprenant et al. (1992) which specifically addresses NDE techniques which have been (or have the potential to be) applied to civil structures and materials.

For discussion, the available methods are grouped under two broad headings: Macro Techniques and Micro Techniques. The former refers to methods which use gross structural motion or force measurements to infer structural condition, while the latter class includes the more traditional NDE techniques designed to isolate flaws through localized testing procedures[6]. There are techniques which fall between these classifications. A good example is the use of imbedded fiber-optic sensors in concrete for strain measurement and crack detection (e.g., Wolff 1990; Huston et al. 1992). These distributed sensors are potentially capable of identifying flaw occurrence and location.

5.2.1 Macro Techniques

Two specific examples of macro-NDE methods are load testing and modal analyses. In both these methods, the structure itself is effectively used as a macro-transducer.

Simplified forms of load testing have been used for many years in the rating process for bridges; recently, more sophisticated versions have been applied to provide data for estimating the actual load-carrying capacity of structures in order to generate more reliable assessments of structure condition. Many of these tests have demonstrated that existing structures indeed possess significantly more capacity than current analytical procedures would suggest (Bakht and Csagoly 1980).

The format of these tests varies greatly, but follow a common theme. The structure is loaded at service load levels or proof tested at higher loads either statically or dynamically

[6]I.e., those that attempt to locate and identify flaws or defects on a scale which is small relative to the overall structure dimensions.

using loaded vehicles, hydraulic rams, or dead weight -- sand bags, concrete blocks, steel ingots[7] and the deflection (using dial gauges, LVDTs and tiltmeters) or strains (strain gauges) and forces (load cells) in individual members is measured (Overman et al. 1987). Based on these load-deformation characteristics, the structure's performance can be evaluated, and estimates of its reliability can be updated (Nowak and Tharmabala 1988).

An alternate method uses the so-called "dynamic signature" of the bridge structure to ascertain the presence of flaws or cracks (Overman et al. 1987; Biswas et al. 1990; Flesch and Kernbichler 1990; Agbabian et al. 1990; Casas Ruis 1990; Hearn 1992; Hearn et al. 1992; O'Leary et al. 1992; Mazurek et al. 1992; Alampalli et al. 1992). Measurements are made of the structure's natural frequencies and modes of vibration, then attempts are made to determine (from shifts in the frequencies or perturbations to the measured mode shapes) when and where a structure may be damaged. These techniques have proven (predictably) rather limited in capability. They are generally capable of identifying the presence of only relatively gross flaws, and are very limited in their ability to determine their location(s). This might be expected, since this "macro-technique" uses integrated structure information, and therefore is limited in its ability to capture "micro-effects."

5.2.2 Micro Techniques

Malhotra (1984) reviews nondestructive methods of concrete testing which are generally used to determine physical, mechanical, or morphological properties of the material. With good models of concrete behavior or well documented correlation studies, it may then be possible to relate the nondestructively measured parameter to the remaining strength of the concrete. Some of the more widely used methods which are truly nondestructive, that is, which leave the material unchanged after the test, are described below.

Surface hardness testing (Akashi and Amasaki 1984) consists of performing standardized impact tests using specified impactors and energy levels, and measuring the size of the resulting indentation and the magnitude of the rebound. While useful under carefully

[7]Alternately, the structure may be pre-deflected and released using a cable or jack system.

controlled circumstances, the impact hammer test has some limitations that prevent it from providing reliable results. The outcome of the test can be affected by extraneous conditions likely to occur in the field. These are the smoothness of the concrete surface, as well as its level of carbonization, moisture condition, and aggregate size and type.

The wavespeed of an ultrasonic pulse propagating through the concrete has been correlated with the uniformity of the mix and can be used to estimate the strength (Naik 1979; Kaplan 1958; Chung and Law 1983). Here also, the relationship to strength is affected by a number of extraneous variables, like age, moisture condition, mix ratio, type of aggregate, and location of reinforcement. In addition to these tests, there are several others which in some way damage a portion of the concrete, either through pullout of an embedded member, breakoff of a cast cylinder, or penetration of a high-velocity projectile (Bungey 1981; Dahl-Jorgensen and Johansen 1984). These cannot be considered truly nondestructive, and most can be performed only once at a particular location.

There are additional NDE methods which are not used to assess strength of concrete, but can determine other characteristics such as the presence of defects. Mutual inductance probes, called cover meters, detect the path of steel reinforcing bars in lightly reinforced sections (Malmberg and Skarendahl 1978). Electrochemical methods have been used to detect active corrosion of reinforcement, where the electrochemical half-cell potential of the reinforcing steel is measured with respect to a reference electrode (Esalante et al. 1982). The method is described in ASTM Standard C 876. While it is useful for determining the rate of corrosion at the time of measurement, that rate is dependent on factors which may change with time. For this reason, an estimate of the integrated effect of the corrosion can generally not be made with the electrochemical technique. Nonetheless, it is useful for detecting active corrosion, and is routinely used for that purpose.

Radiographic shadow graphs have been attempted to detect and assess reinforcing steel, voids and segregation, grouting of post-tensioning ducts, and to reveal cracks. To penetrate concrete structures of interest requires high energy, high flux x-rays. Devices which can generate such x-ray beams, electron accelerators in conjunction with rotating targets, are not normally portable. In view of that, most efforts have concentrated on the use of radioactive sources to provide highly penetrating gamma rays (Honig 1984). Still, there are the drawbacks of relatively long exposure times, high cost of film, and the special precautions and cost associated with the handling of highly radioactive materials.

Related to the radiographic methods are techniques which exploit the moderating effect of water on fast neutrons. These methods are also difficult and cumbersome to apply. Microwave absorption has been used to determine the moisture content of concrete (Cantor and Kneeter 1981). Due to the large dielectric relaxation of polar molecules at microwave frequencies, the presence of water can be detected and quantified by observing the attenuation of a reflected microwave beam. The main advantage of this test is that it can be performed with the test equipment in motion. Thermography has been attempted with limited success to detect delaminated concrete slabs or cracks. Differences in emittance with the presence of defects should lead to detectable contrast in a thermal image. The effects of moisture and other concrete conditions lower the test's reliability.

In the acoustic emission technique, the concrete structure is instrumented with transducers which detect ultrasonic signals emitted during the growth of cracks (Nielsen and Griffin 1977). A major difficulty with the application of this method is the problem of discrimination of crack growth events from other noise sources. For continuous structural monitoring, the integrity of the transducer bond over time is an important issue. The character of the bond between the ultrasonic transducer and the structure strongly affects the nature of the received signal.

The impact-echo method of evaluating concrete has been used to determine mechanical properties and to detect distributed and discrete defects. Developed at the National Institute for Standards and Technology (Carino et al. 1986; Sansalone and Carino 1989). The improved impact-echo technique consists of dropping a steel ball from a known height onto the concrete surface and monitoring the response of the concrete structure with a displacement transducer at a remote location. From the echoes recorded at the transducer and their frequency spectrum, an assessment of the concrete condition can be made, and defects like delaminations 0.5 m in diameter, or larger, can be reliably detected. Limitations of the impact-echo method are the variability of the input signal, which depends on the concrete surface conditions or presence of an overlayer, the uncontrolled distribution of acoustic power across the frequency spectrum, and the point-source diffraction of the input pulse. These latter two features in particular result in only a small fraction of the input acoustic power density remaining in the direction and frequency band of interest for the nondestructive test at hand, thus limiting the detection limits and resolution of the technique.

Pulse-echo ultrasonic testing of concrete (Thornton and Alexander 1987; Alexander and Thornton 1989; Bradfield and Gatfield 1964; Claytor and Ellingson 1985; Mailer 1972) has also been studied
over the past twenty years, although most applications have been limited to the measurement of pavement thickness. Various means have been used to overcome the drawbacks of this method. Because of the relatively large attenuation of concrete (0.3-30 dB/m at 150 kHz (Carino and Sansalone 1984)), frequencies are generally limited to about 200 kHz. At these low frequencies beam forming can be a challenge. In order to produce a directed ultrasonic beam, a transducer width to wavelength ratio of about three or greater is needed. For concrete, with an average compressional wavespeed of 4 km/sec, this requirement, at 150 kHz, implies a minimum transducer width of 8 cm, or about 3 inches. Such large transducers can be difficult to couple to the concrete surface, and they are generally not efficient for detection of reflected acoustic waves. Normally, a second transducer is used as a receiver. Low-frequency transducers are also inherently narrow-band devices, since a 50% bandwidth gives a useable range of only 100 to 200 kHz for a 150 kHz center frequency transducer. To resolve small targets in time, one conventionally requires a high-frequency transducer band edge of the inverse of the propagation time separation for the targets. Moreover, the efficiency of the transducer is reduced by the viscous damping introduced to increase the bandwidth. These competing requirements complicate the problem of ultrasonic defect detection in highly inhomogeneous material like concrete and demand innovative approaches to circumvent the problems.

6. RESEARCH NEEDS

It is appropriate at this stage to reiterate some of the issues raised in this paper, as they should be considered during the identification of suitable research agendas.

Research in NDE of civil structures must be performed within the context of the overall assessment problem, i.e., the establishment of fitness-for-service criteria, and how these might be assessed using available and new technologies. NDE techniques used for this purpose should meet (at a minimum) the following criteria:

1. Be usable under difficult field conditions.
2. Minimize the effects of operator influence (i.e., not require highly trained personnel).
3. Be effective with badly deteriorated surfaces.
4. Be economical.
5. Focus on establishing *structural* parameters in situ.

As noted in the introduction, detailed assessment of the research needs is the goal of the workshop, i.e., they will be identified and prioritized by the workshop attendees. The reader is referred to the accompanying breakout session summary for enumeration of these issues.

7. SUMMARY

The identification of degradation and flaws in concrete bridges is a formidable problem. Yet it is one of prime importance, as these structures form an essential part of the nation's transportation infrastructure. Nondestructive evaluation techniques, coupled with an appropriate data collection and statistically based assessment framework, are poised to contribute significantly to the solution of this problem.

To effect this contribution, coordination of research efforts is required, and the multidisciplinary nature of the effort must be recognized and optimized. It is hoped that this forum, in providing an opportunity to bring the NDE and bridge engineering communities together, will indeed begin the process of identification of research needs, and spur the focusing of future directions in this promising field.

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Table 1. Status of U.S. Bridge Inventory

State	Total bridges	Structurally deficient	Functionally obsolete	Total deficient	Percent deficient	Rank
Alabama	15,534	3,949	3,602	7,551	49	13
Alaska	800	86	27	111	14	49
Arizona	5,623	160	252	412	7	51
Arkansas	13,017	1,596	4,225	5,821	45	17
California	22,261	1,666	4,055	5,721	26	37
Colorado	7,428	2,208	460	2,668	36	27
Connecticut	3,749	2,394	1,368	2,401	64	2
District of Columbia	237	48	1	49	21	39
Delaware	738	79	96	175	24	43
Florida	10,188	610	1,605	2,215	22	41
Georgia	14,226	3,520	2,518	6,038	42	21
Hawaii	1,043	115	161	276	26	37
Idaho	3,745	560	514	1,074	29	33
Illinois	25,428	5,313	2,042	7,355	29	33
Indiana	17,517	3,807	3,939	7,656	44	9
Iowa	25,865	6,040	6,336	12,376	48	15
Kansas	25,648	5,386	7,347	12,733	50	23
Kentucky	12,591	2,207	5,252	7,459	59	5
Louisiana	14,139	3,959	2,443	6,402	45	17
Maine	2,583	436	331	767	30	32
Maryland	4,574	703	1,169	1,872	41	23
Massachusetts	4,964	1,714	209	1,923	39	26
Michigan	10,581	2,628	683	3,311	31	31
Minnesota	12,994	1,911	1,787	3,698	28	35
Mississippi	16,994	6,421	2,563	8,984	53	10
Missouri	23,682	12,347	2,718	15,065	64	2
Montana	4,632	495	2,240	2,735	59	5
Nebraska	15,843	7,636	1,158	8,794	56	8
Nevada	1,073	50	109	159	15	49
New Hampshire	2,572	522	603	1,125	44	19
New Jersey	5,997	1,352	752	2,104	35	28
New Mexico	3,439	410	334	744	22	41
New York	17,326	10,409	1,403	11,812	68	1
North Carolina	16,115	1,107	7,382	8,489	53	10
North Dakota	5,283	1,959	582	3,041	58	7
Ohio	29,180	4,494	1,504	5,998	21	43
Oklahoma	22,981	8,229	4,677	12,906	56	8
Oregon	6,608	577	558	1,135	17	47
Pennsylvania	22,457	5,990	2,917	8,907	40	24
Rhode Island	702	98	38	136	19	46
South Carolina	8,886	939	836	1,775	20	45
South Dakota	6,822	1,660	1,530	3,190	47	16
Tennessee	18,547	4,366	3,023	78,389	40	24
Texas	44,314	6,572	8,581	15,153	34	29
Utah	2,543	262	96	358	14	49
Vermont	2,665	503	808	1,311	49	13
Virginia	12,652	3,933	1,610	4,284	34	29
Washington	6,898	920	941	1,861	27	36
West Virginia	6,513	2,795	1,196	3,991	61	4
Wisconsin	12,963	3,978	1,455	5,433	42	21
Wyoming	2,826	320	356	676	24	39
U.S. TOTAL	577,710	135,826	102,531	238,357	41	

Source: Federal Highway Administration, U.S. DOT

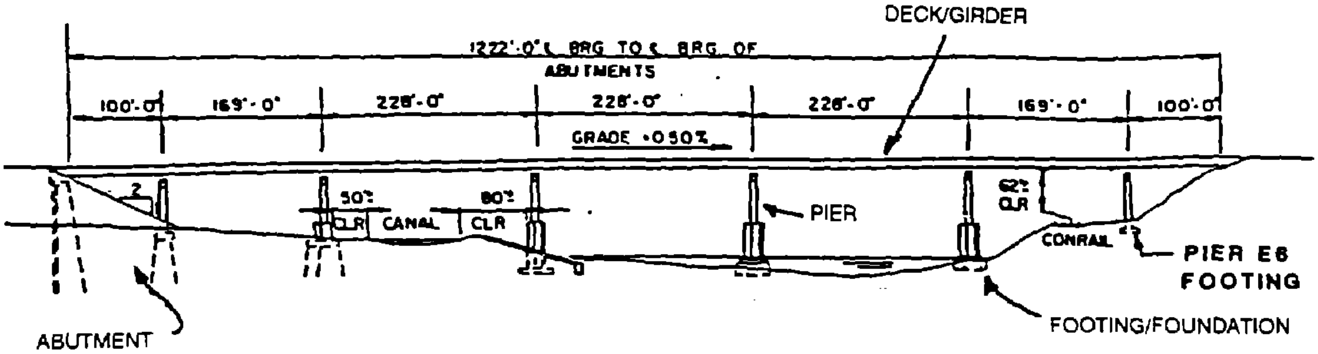

Figure 1. Typical Bridge Components

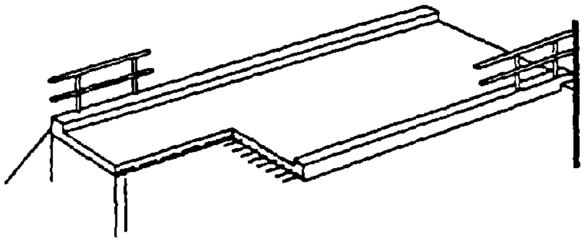

Figure 2a. Slab Bridge (Winter and Nilson 1979)

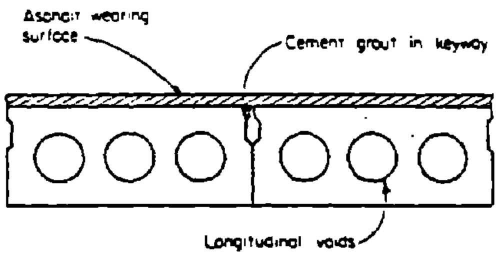

Figure 2b. Voided Slab Section (Winter and Nilson 1979)

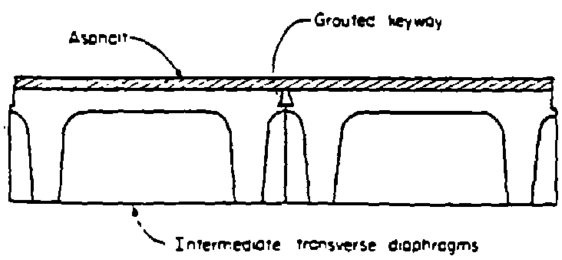

Figure 3. Precast Channel Slab Bridge (Winter and Nilson 1979)

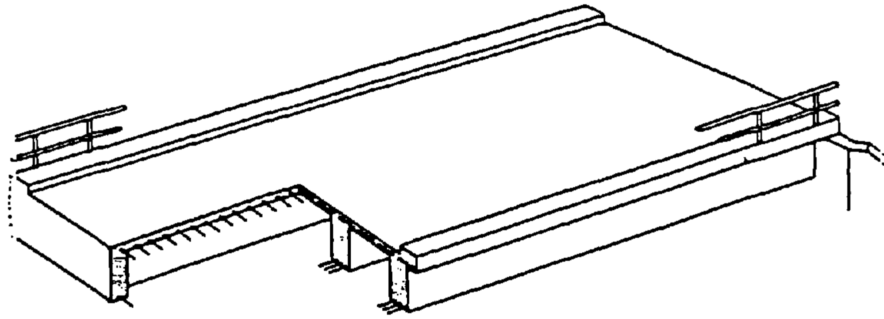

Figure 4. Reinforced Concrete Deck-Girder Bridge (Winter & Nilson 1979)

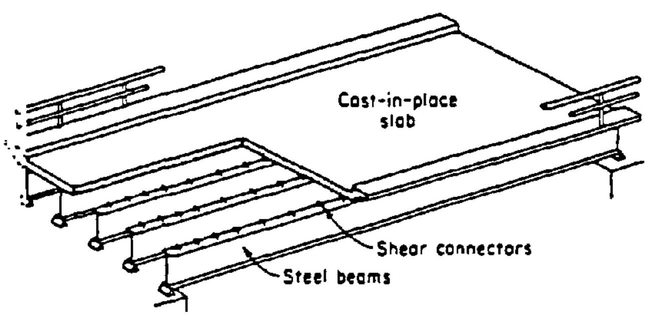

Figure 5. Composite Steel-Concrete Bridge (Winter & Nilson 1979)

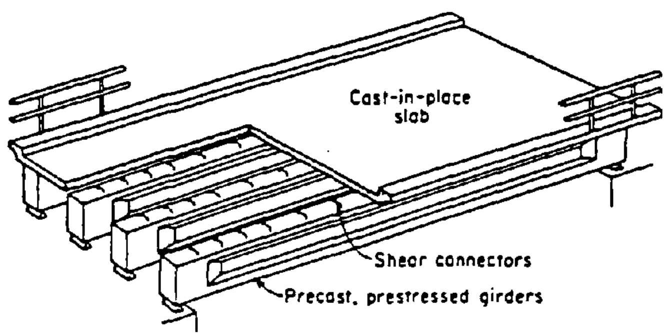

Figure 6. Composite Prestressed-Concrete Bridge (Winter & Nilson 1979)

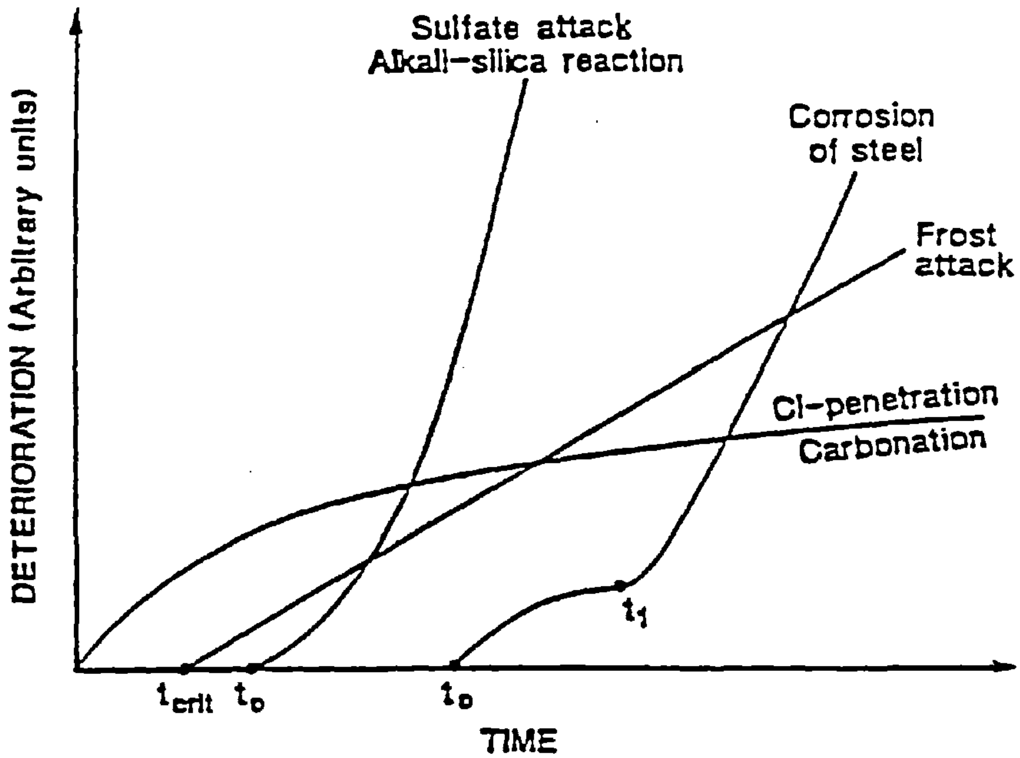

Figure 7. Material Degradation (Clifton & Knab 1989)

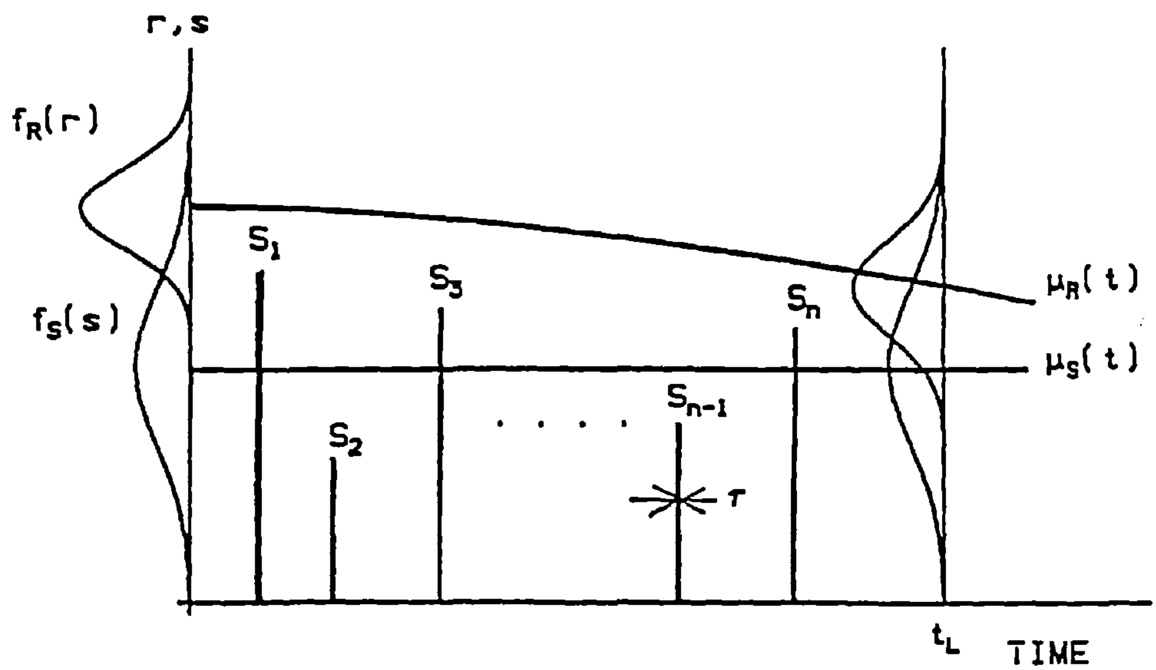

Figure 8. Schematic Representation of Load Process and Degradation of Resistance

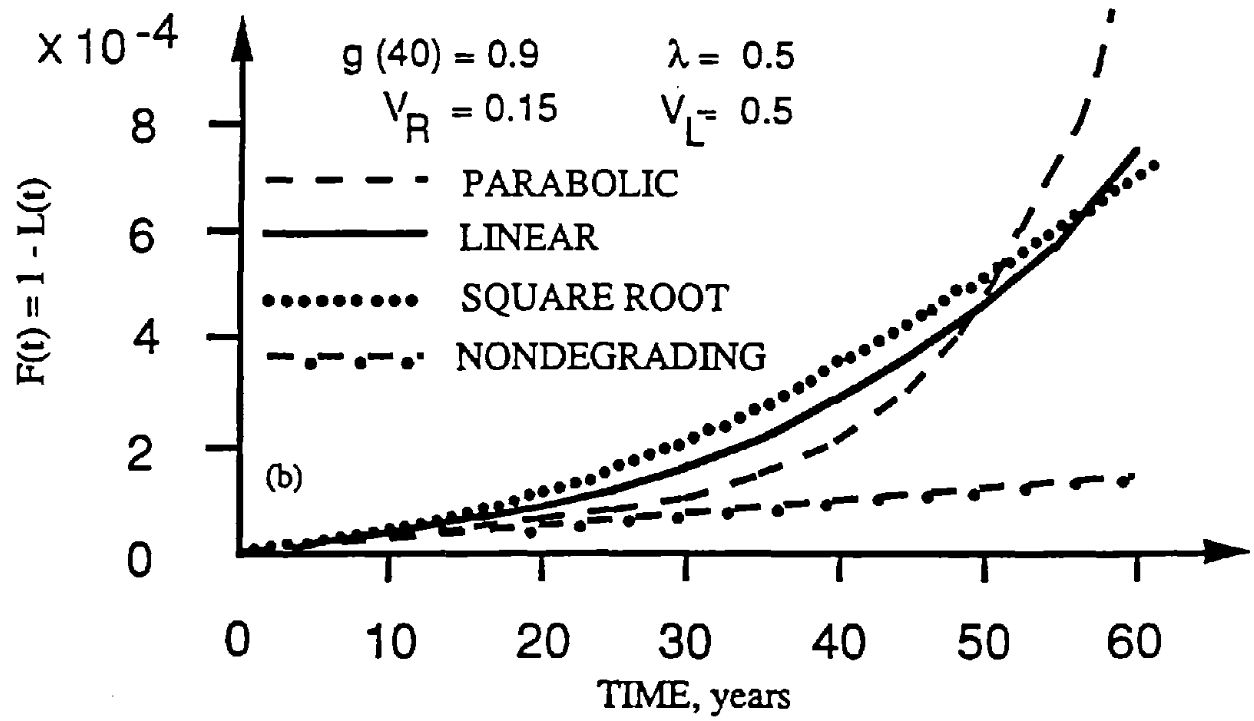

Figure 9. Dependence of Single Component Failure Probability on Degradation Model: D+L

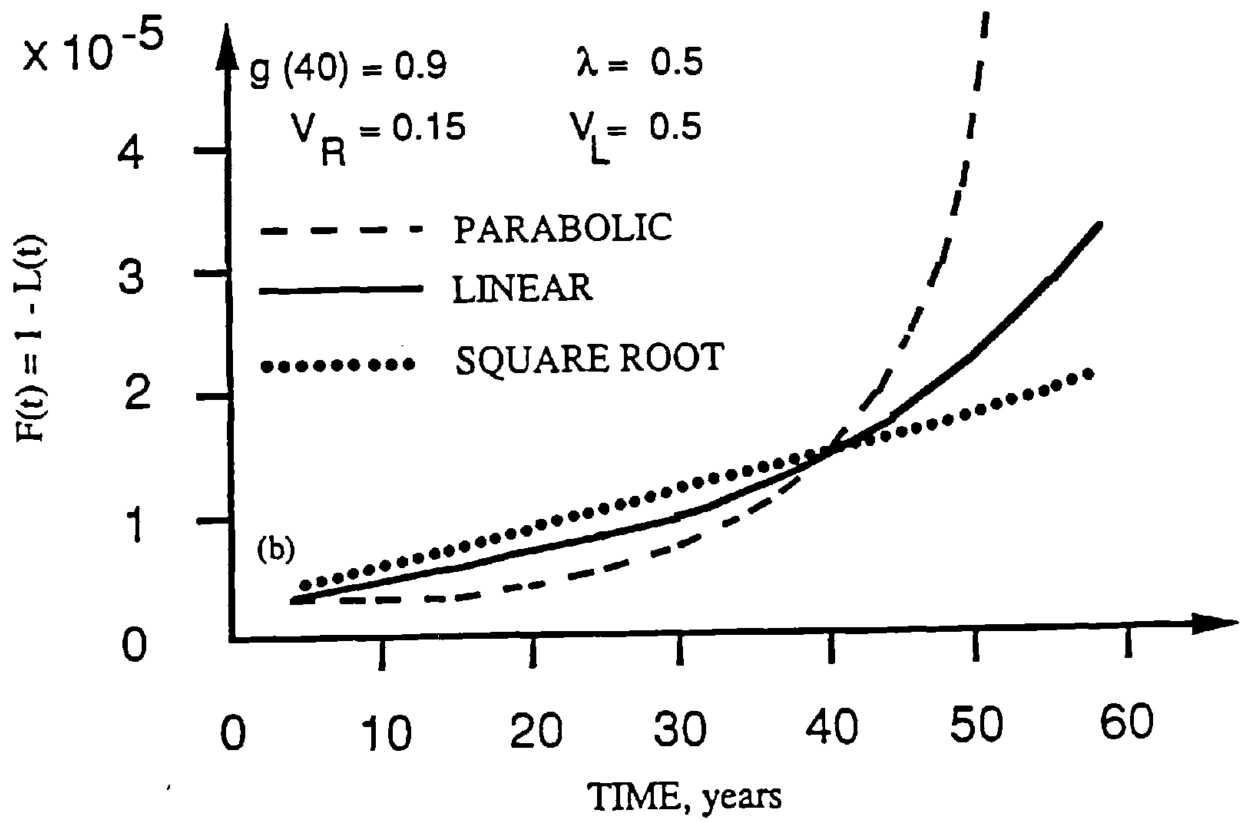

Figure 10. Hazard Function of Single Component

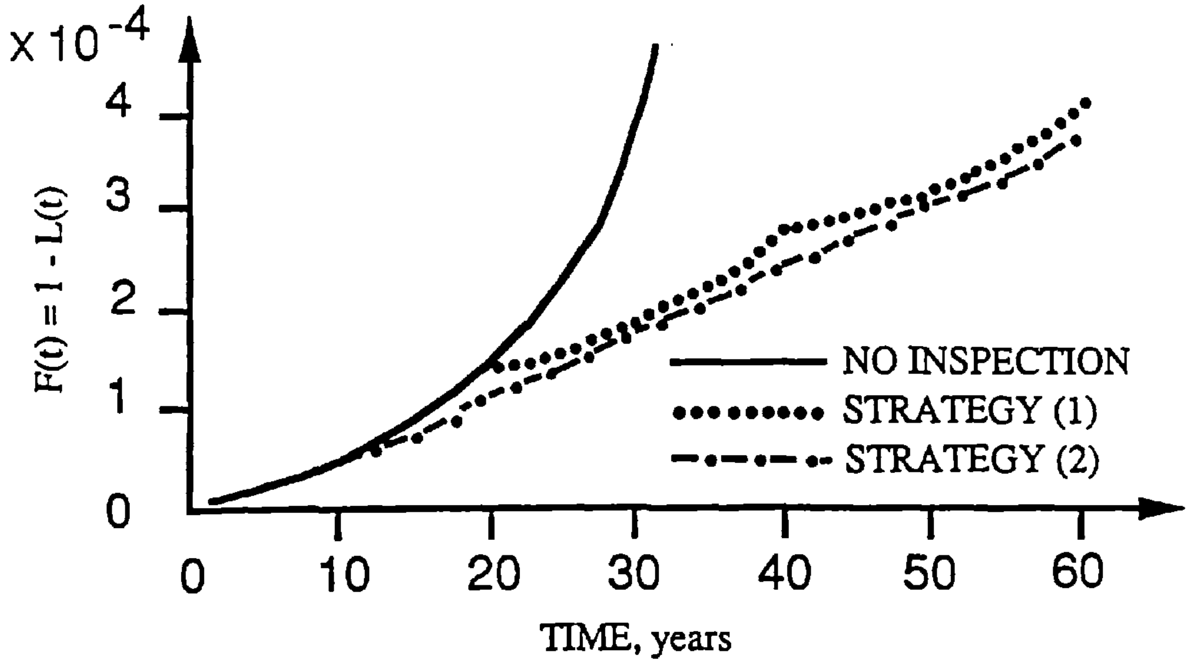

Figure 11. Failure Probability with Repair

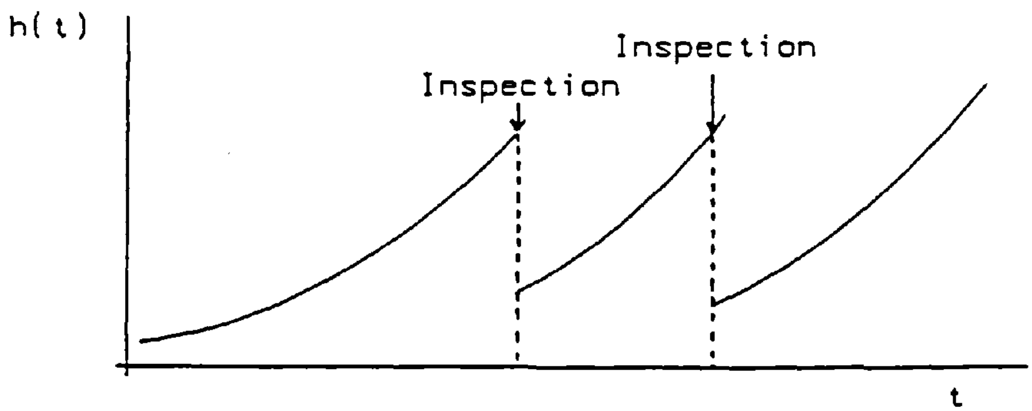

Figure 12. Role of Inspection in Controlling Hazard Function

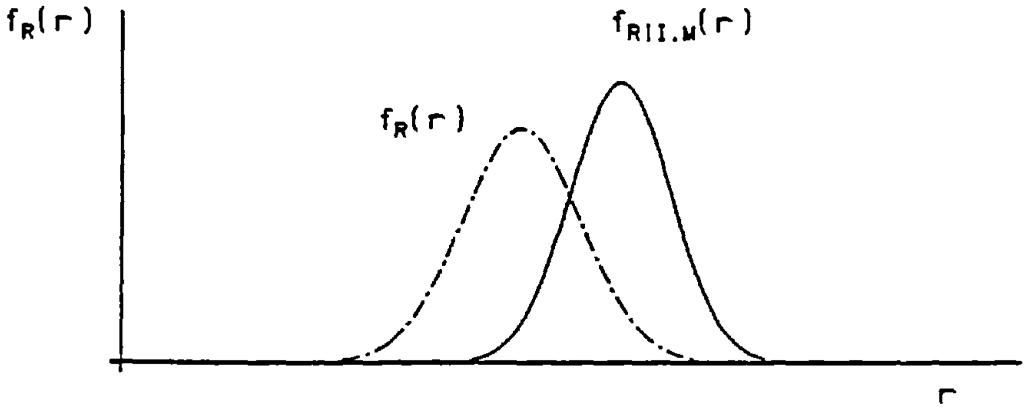

Figure 13. Role of Inspection on Strength Distribution

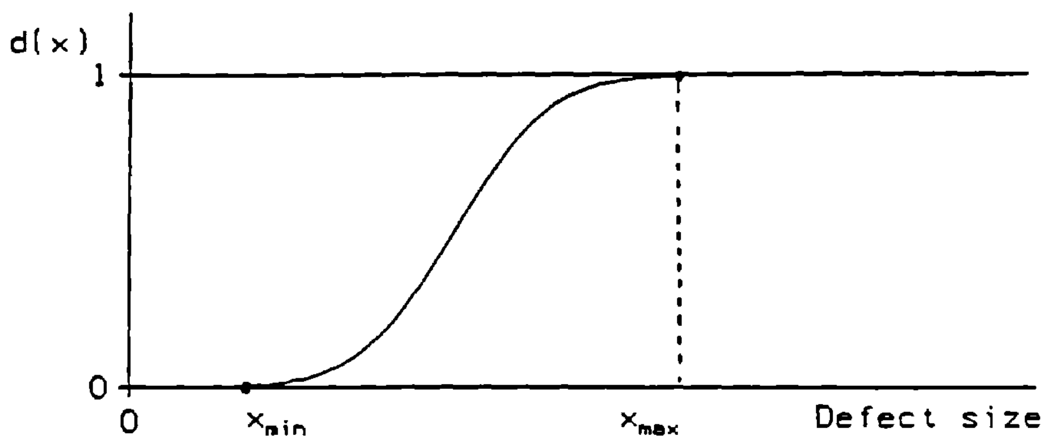

Figure 14. Defect Detectability Function

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Nondestructive Testing for Assessing Wood Members in Structures

A Review

Robert J. Ross, Supervisory General Engineer
Forest Products Laboratory, Madison, Wisconsin

Roy F. Pellerin, Professor of Structural Engineering
Washington State University, Pullman, Washington

Executive Summary

The USDA Forest Service, Forest Products Laboratory (FPL), and Washington State University (WSU) have been actively developing nondestructive testing (NDT) techniques for wood products for more than 30 years. Their individual and combined efforts of research and technology transfer activities have yielded a variety of NDT tools and techniques that are commonly used by manufacturers and users of forest products throughout the world.

Recently, individuals and organizations have shown considerable interest in the use of NDT for assessing the performance of wood members in structures. Both the FPL and WSU have received numerous requests for background information that illustrates use of NDT techniques for in-place member assessment. Questions are frequently asked about fundamental NDT concepts and about previous NDT research that might be extended to a particular application.

We prepared this report to provide a synthesized information base to aid in addressing such requests. This report is a compilation of various published research and application efforts that have focussed on NDT of wood products. The report begins by examining fundamental concepts for NDT of wood. It then reviews pertinent laboratory investigations designed to explore fundamental concepts and presents several examples of how to apply these concepts to in-place assessment of wood members. Recommendations are also given for future in-place assessment NDT research.

Introduction

By definition, nondestructive materials evaluation is the science of identifying physical and mechanical properties of a piece of material without altering its end-use capabilities. Such evaluations rely upon nondestructive testing (NDT) techniques to provide accurate information pertaining to the properties, performance, or condition of the material in question.

Historically, the wood products community has developed and used NDT techniques almost exclusively for sorting or grading structural products. Two excellent examples are machine stress rating (MSR) of lumber and ultrasonic grading of veneer. As currently practiced in North America, MSR couples visual sorting criteria with nondestructive measurements of the stiffness of a piece of lumber to assign it to an established grade (Galligan and others 1977). Similarly, laminated veneer manufacturing facilities use stress wave NDT techniques to sort incoming veneer into strength classes prior to processing into finished products. Veneers are assigned to strength categories, which are established through empirical relationships between stress wave velocity and strength, based on the velocity at which an induced stress wave travels through the veneer (Sharp 1985).

However, a need also exists for NDT techniques to be used in the evaluation of wood in structures. This need is expanding because an increasing amount of resources are being devoted to repair and rehabilitation of existing structures rather than to new construction. As more resources are devoted to repair, an increasing emphasis must be placed on the in-place assessment of structures. This, in turn, requires accurate, cost-effective NDT techniques.

This report presents a review of literature on NDT techniques used for in-place evaluation of wood in structures. Reports of work utilizing NDT techniques for in-place evaluation of wood in structures are also discussed.

Fundamental Hypothesis

Nondestructive testing techniques for wood differ greatly from those for homogeneous, isotropic materials such as metals, plastics, and ceramics. In such nonwood-based materials, whose mechanical properties are known and tightly controlled by manufacturing processes, NDT techniques are used only to detect the presence of discontinuities, voids, or inclusions. However, in wood, these irregularities occur naturally and may be further induced by degradative agents in the environment. Therefore, NDT techniques for wood are used to measure how natural and environmentally induced irregularities interact in a wood member to determine its mechanical properties.

This concept led researchers to vigorously examine several NDT techniques for grading structural lumber and evaluating the quality of laminated materials (Bell and others 1950; Galiginaitis and others 1954; Jayne 1955, 1959; James 1959; Hoyle 1961b; McKean and Hoyle 1962; Senft and others 1962). Two significant developments evolved from their efforts: MSR of lumber, and perhaps more significant, the evolution of a hypothesis based on fundamental material properties for establishing relationships between measurable NDT parameters and static mechanical properties.

The fundamental hypothesis for NDT of wood materials was initiated by Jayne (1959). He proposed that the energy storage and dissipation properties of wood materials, which can be measured nondestructively by using a number of NDT techniques, are controlled by the same mechanisms that determine the static behavior of such material. As a consequence, useful mathematical relationships between these properties and static elastic and strength behavior should be attainable through statistical regression analysis.

To elaborate on Jayne's (1959) hypothesis, consider how the microscopic structure of clear wood affects its static mechanical behavior and energy storage and dissipation properties. Clear wood is a composite material composed of many tube-like cells cemented together. At the microscopic level, energy storage properties are controlled by orientation of the cells and structural composition, factors that contribute to static elasticity and strength. Such properties are observable as frequency of oscillation in vibration or speed-of-sound transmission. Conversely, energy dissipation properties are controlled by internal friction characteristics, which bonding behavior between constituents contributes to significantly. Rate of decay of free vibration or acoustic wave attenuation measurements are frequently used to observe energy dissipation properties in wood and other materials.

Statistical regression analysis methods are used to establish mathematical relationships between NDT parameters and performance characteristics. As shown in Figure 1, the closer data are grouped around the regression line and the lower the variability, the more successful an NDT parameter is at predicting performance. In the literature we reviewed, most researchers reported on the quality of an NDT parameter in terms of a correlation coefficient r. Correlation coefficients can range from -1 to 1. A correlation coefficient nearing 1 suggests a strong positive relationship, and a coefficient near 0.7 indicates a positive relationship. A correlation coefficient of zero reveals that no relationship exists, positive or negative.

NDT Techniques

The following sections briefly describe several techniques used to nondestructively evaluate wood-based materials.

Static Bending Techniques

Measuring modulus of elasticity (MOE) of a member by static bending techniques is the foundation of MSR of lumber. As currently employed for MSR, the relatively simple measurement involves utilizing the load-deflection relationship of a simply supported beam loaded at its midspan (Fig. 2). Modulus of elasticity can be computed directly by using equations derived from fundamental mechanics of materials and used to infer strength.

Transverse Vibration Techniques

Transverse vibration techniques have received considerable attention for NDT applications. To illustrate these methods, an analogy can be drawn between the behavior of a vibrating beam and the vibration of a mass that is attached to a weightless spring and internal damping force (Fig. 3). In Figure 3, mass M is supported from a rigid body by a weightless spring whose stiffness is denoted by K. Internal friction or damping is represented by the dashpot D. A forcing function equaling $P_0 \sin \omega t$ or zero is applied for forced and free vibration, respectively. When M is set into vibration, its equation of motion can be expressed by the following:

$$M(\frac{d^2x}{dt^2}) + D(\frac{dx}{dt}) + Kx = P_0 \sin \omega t \qquad (1)$$

Equation (1) can be solved for either K or D.

A solution for K will lead to an expression for MOE where

$$\text{MOE} = \frac{f_r^2 W L^3}{12.65 I g} \qquad (2)$$

for a beam freely supported at two nodal points and

$$\text{MOE} = \frac{f_r^2 W L^3}{2.46 I g} \qquad (3)$$

for a beam simply supported at its ends.

In Equations (2) and (3),

MOE is dynamic modulus of elasticity (lb/in^2 (Pa)),

f_r resonant frequency (Hz)

W beam weight (lb (kg·g)),

L beam span (in. (m)),

I beam moment of inertia (in^4 (m^4)), and

g acceleration due to gravity (386 in/s^2 (9.8 m/s^2)).

Solving Equation (1) for D leads to an expression of the internal friction or damping component. The logarithmic decrement of vibrational decay δ is a measure of internal friction and can be expressed in the form (for free vibrations)

$$\delta = \frac{1}{(n-1)} \ln \frac{A_1}{A_n} \qquad (4)$$

where A_1 and A_n are the amplitudes of two oscillations $n-1$ cycles apart (Fig. 4).

For forced vibrations,

$$\delta = \frac{\pi \Delta f}{f_r} \frac{1}{\sqrt{(A_r/A)^2 - 1}} \qquad (5)$$

where

Δf is the difference in frequency of two points of amplitude A on each side of a resonance curve,

f_r the frequency at resonance, and

A_r the amplitude at resonance (Fig. 4b).

Sharpness of resonance Q is frequently used to measure damping capacity; Q is defined as the ratio of f_r/f. Note that if the value $0.707A_r$ (half-power point method) is substituted for A in Equation (5), the equation reduces to

$$\delta = \frac{\pi \Delta f}{f_r} \qquad (6)$$

and

$$Q = \frac{\pi}{\delta} \qquad (7)$$

Stress Wave Techniques

Several techniques that utilize stress wave propagation have been researched for use as NDT tools. Speed-of-sound transmission and attenuation of induced stress waves in a material are frequently used as NDT parameters.

To illustrate these techniques, consider application of one-dimensional wave theory to the homogeneous viscoelastic bar (Fig. 5). After an impact hits the end of the bar, a wave is generated. This wave immediately begins moving down the bar as particles at the leading edge of the wave become excited, while particles at the trailing edge of the wave come to rest. The wave moves along the bar at a constant speed, but its individual particles have only small longitudinal movements as a result of the wave passing over them. After traveling the length of the bar, this forward-moving wave impinges on the free end of the bar, is reflected, and begins traveling back down the bar.

Energy is dissipated as the wave travels through the bar; therefore, although the speed of the wave remains constant, movement of particles diminishes with each successive passing of the wave. Eventually all particles of the bar come to rest.

Monitoring the movement of a cross section near the end of such a bar in response to a propagating stress wave results in waveforms that consist of a series of equally spaced pulses whose magnitude decreases exponentially with time (Fig. 6). The propagation speed C of such a wave can be determined by coupling measurements of the time between pulses Δt and the length of the bar L by

$$C = \frac{2L}{\Delta t} \qquad (8)$$

The MOE can be computed using C and the mass density of the bar ρ:

$$\mathrm{MOE} = C^2\rho \quad (9)$$

Wave attenuation can be determined for the rate of decay of the amplitude of pulses using Equation (4) for logarithmic decrement.

Note that wave attenuation calculated using this formula is highly dependent upon characteristics of the excitation system used. Thus, results reported by various researchers cannot be directly compared because several excitation systems were employed. As their results show, energy loss characteristics as measured by stress wave techniques provide useful information pertaining to the performance of wood-based materials.

A more rigorous treatise on the measurement of energy loss by stress wave techniques is presented by Kolsky (1963). In general, a more appropriate method for evaluating energy loss would be to determine the quantity of energy imparted into a member and the corresponding rate of loss of energy. Loss of energy would be calculated using an integral of a waveform, as is done for determining the energy emitted during acoustic emission testing of materials (Harris and others 1972). This is defined as the root mean square (RMS) value.

Wood is neither homogeneous nor isotropic; therefore, the usefulness of one-dimensional wave theory for describing stress wave behavior in wood could be considered dubious. However, several researchers have explored application of the theory by examining actual waveforms resulting from propagating waves in wood and wood products and have found that one-dimensional wave theory is adequate for describing wave behavior. For example, Bertholf (1965) found that the theory could be used to accurately predict dynamic strain patterns in small wood specimens. He verified predicted stress wave behavior with actual strain wave measurements and also verified dependence of propagation velocity on the MOE of clear wood. Ross (1985) examined wave behavior in both clear wood and wood-based composites and observed excellent agreement with one-dimensional theory. Similar results were obtained with clear lumber in tests conducted by Kaiserlik and Pellerin (1977).

An interesting series of experiments designed to explore wave behavior in lumber was also conducted by Gerhards (1981, 1982). He observed changes in the shape of a wave front in lumber containing knots and cross grain by measuring the change in wave speed in the vicinity of such defects. He concluded that a stress wave traveling in lumber containing knots and cross grain does not maintain a planar wave front.

One commonly used technique that employs stress wave NDT technology utilizes simple time-of-flight-type measurement systems to determine speed-of-wave propagation (Figs. 7,8). In these measurement systems, a mechanical or ultrasonic impact is used to impart a longitudinal wave into a member. Piezoelectric sensors are placed at two points on the member and used to sense passing of the wave. The time it takes for the wave to travel between sensors is measured and used to compute wave propagation speed.

Several research projects designed to examine application of one-dimensional theory to wave propagation in clear wood, lumber, and veneer have been conducted using this type of measurement. These projects examined relationships between MOE values obtained from stress wave measurements and those measured using static testing techniques. Note the strong correlative MOE relationships found in these research projects (Table 1).

Considerable research activity has focused on development of techniques to measure stress wave attenuation in wood products. For example, Ross and Pellerin (1988) used an inexpensive velocity meter to measure wave attenuation. Others (Beall 1987, Patton–Mallory and De Groot 1989) examined coupling acoustic emission (AE) and ultrasonic techniques to measure wave attenuation.

Acoustic emission techniques have also been extensively researched for application to wood-based materials. These techniques rely upon the application of stress to a member to generate a stress wave. An excellent review of AE techniques and research related to their application to wood-based materials is presented by Beall (1987).

Other Techniques

Several other NDT techniques have been investigated for use with wood. For example, the attenuation of x-rays has been investigated for detecting internal voids in wood (Mothershead and Stacey 1965) and for inspecting utility poles and trees (Monro and others 1990).

Screw withdrawal (Talbot 1982) and pick- or probing-types tests have also been examined. These inexpensive techniques provide information about a member at a point and are consequently of limited value for inferring strength for large members. However, they are useful for detecting surface damage of members.

The Pilodyn test is also used to detect surface damage. The Pilodyn instrument consists of a spring-loaded pin device that drives a hardened steel pin into the wood.

Depth of pin penetration is used as a measure of degree of degradation (Hoffmeyer 1978).

Laboratory Verification of Fundamental Hypothesis

Several research organizations have examined application of fundamental concepts under laboratory conditions. The following sections summarize results presented by these organizations.

Clear Wood and Lumber Products

Initial laboratory studies to verify the fundamental hypothesis were conducted with clear wood and lumber products using a variety of NDT techniques. For example, considerable research activity was conducted in the early 1960s to examine relationships between the static bending MOE and ultimate strength of softwood dimension lumber. Results obtained from various projects designed to examine this relationship are summarized in Tables 2 to 4. Note that useful correlative relationships were found between MOE and the bending, compressive, and tensile strengths of dimension lumber obtained from various softwood species.

Research using transverse vibration and stress wave techniques is summarized in Table 5. Jayne (1959) designed and conducted one of the first studies that utilized transverse vibration NDT techniques for evaluating the strength of wood. He was successful in demonstrating a relationship between energy storage and dissipation properties, measured by forced transverse vibration techniques, and the static bending properties of small, clear wood specimens. He utilized an experimental setup similar to that illustrated in Figure 9. With this setup, Jayne was able to determine the resonant frequency of a specimen from a frequency response curve. In addition, sharpness of resonance (energy loss) was obtained using the half-power point method. Pellerin (1965a,b) verified the hypothesis using free transverse vibration techniques on dimension lumber and glulam timbers with the apparatus shown in Figure 10. After obtaining a damped sine waveform for a specimen (Fig. 3), he analyzed it utilizing equations for MOE and logarithmic decrement. Measured values of MOE and logarithmic decrement were then compared to static MOE and strength values. O'Halloran (1969) used a similar apparatus and obtained comparable results with softwood dimension lumber.

Kaiserlik and Pellerin (1977) furthered the hypothesis by using stress wave techniques to evaluate the tensile strength of a small sample of clear lumber containing varying degrees of slope of grain (Fig. 11). They utilized the one-dimensional wave Equation (9) to compute MOE and the equation presented by Pellerin (1965b) for logarithmic decrement.

Wood-Based Composite Materials

The fundamental hypothesis was verified using stress wave techniques on wood-based composites (Suddarth 1965, Pellerin and Morschauser 1974, Ross 1984, Fagan and Bodig 1985, Vogt 1985, and Ross and Pellerin 1988) (Table 6). Pellerin and Morschauser (1974) used the setup in Figure 7 to show that stress wave speed, a measure of energy storage properties, could be used to predict the flexural behavior of underlayment grade particleboard. Ross (1984) and Ross and Pellerin (1988) revealed that wave attenuation, a measure of energy dissipation properties, is sensitive to bonding characteristics and is a valuable NDT parameter that contributes significantly to the prediction of tensile and flexural mechanical behavior of wood-based particle composites. Vogt (1985) furthered the application of the hypothesis to wood-based fiber composites. In an additional study, Vogt (1986) found a strong relationship between internal bond and stress wave parameters of particle and fiber composites. Suddarth (1965) verified the hypothesis by using forced transverse vibration techniques to locate poorly bonded or debonded areas in wood components for missiles.

Biologically Degraded Wood

Verification of the hypothesis with wood subjected to different levels of deterioration by decay fungi, which adversely effect the mechanical properties of wood and are frequently found in wood structures, has been limited to studies that have employed only energy storage parameters (Table 7). Wang and others (1980) found that wood decay significantly affected the frequency of oscillation of small, eastern pine, sapwood, cantilever bending specimens (Fig. 12). Pellerin and others (1985) showed that stress wave speed could be successfully used to monitor the degradation of small clear-wood specimens exposed to brown-rot fungi. They showed a strong correlative relationship between stress wave speed and parallel-to-grain compressive strength of exposed wood. Rutherford and others (1987) showed similar results. They also revealed that MOE perpendicular to the grain, measured using stress wave NDT techniques, was significantly affected by degradation from brown-rot decay and could be used to detect incipient decay. Chudnoff and others (1984) reported similar results from experiments that utilized an ultrasonic measurement system (Fig. 8) and several hardwood and softwood species. Patton-Mallory and

De Groot (1989) reported encouraging results from a fundamental study dealing with the application of acousto-ultrasonic techniques (Fig. 13). Their results showed that energy loss parameters may provide useful additional information pertaining to early strength loss from incipient decay caused by brown-rot fungi.

Acoustic emission techniques were also investigated for use in decay detection. Utilizing a small sample of clear, white fir specimens infected with brown-rot fungi, Beall and Wilcox (1986) were able to show a relationship between selected AE parameters and radial compressive strength (Fig. 14).

In-Place Assessment of Wood Members

Several organizations have published research results on the use of NDT techniques for in-place evaluation of wood members (Table 8). The following summarizes research conducted on the use of several NDT techniques for such evaluations.

Static Bending Techniques

Measuring flexural MOE by static bending techniques has been successfully employed to grade lumber by using machines that approximate simply supported boundary conditions. Such machines consistently maintain these conditions. However, an in-place environment yields boundary conditions that may vary considerably in even the simplest structure. Consequently, application of this technique for in-place assessment of wood members has been limited.

Abbott and Elcock (1987) developed an in-place NDT technique for measuring the stiffness of in-service poles (Fig. 15). A bending load was applied to individual poles above the ground line. Load and resulting deflections were recorded and used to compute flexural stiffness. From these measurements, inferences pertaining to pole strength were made, and predicted and actual values were compared.

Transverse Vibration Techniques

Transverse vibration techniques are also significantly influenced by boundary conditions. Most researchers conducting laboratory studies with this technique devote considerable time to insuring that simple end conditions are attained. As discussed previously, such conditions frequently do not exist with wood members in structures. Consequently, use of this technique has also been limited for in-place evaluations.

Murphy and others (1987) developed a technique based on transverse vibration NDT techniques for evaluating wood poles. Their technique involved measuring the vibrational response of a pole after it is tapped by a rubber mallet. Resonant frequency of the pole was identified and used to infer pole strength.

Stress Wave Techniques

Longitudinal stress wave NDT techniques have also been investigated by researchers for assessing wood members in structures. The influence that boundary conditions have on speed-of-sound transmission measurements has been shown to be significantly less than that for static bending or transverse vibration techniques. Thus, many researchers have examined longitudinal stress wave NDT techniques for in-place assessment of wood members. The following briefly describes stress wave NDT techniques that have been used in projects.

Eighteenth Century Mansion

Lee (1965) was one of the first to examine use of stress wave techniques for in-place evaluation. He assessed the roof structure of an 18th century mansion, using an ultrasonic impact and measurement system similar to that illustrated in Figure 8. He measured propagation speed of stress waves in wood members both parallel and perpendicular to the grain. To obtain an estimate of strength loss, sections from purlins were evaluated statically in a laboratory, and a chart relating stress wave velocity and strength was prepared. Strength of the remaining timbers was then inferred.

University Football Stadium

Washington State University's football stadium, Pullman, Washington, was also inspected using stress wave NDT techniques. This stadium was originally constructed in the 1930s; the north and south grandstands were replaced after a fire in the 1960s. The portion of the stadium that was inspected for its structural integrity in the early 1980s was the horseshoe section that joined the north and south grandstands. This horseshoe section was part of the original stadium and was constructed from large solid-sawn timbers. The reason for inspection was that large crowds were anticipated for the upcoming football season, thus requiring use of the horseshoe section. The university administration hired consulting engineers to inspect and assess the structural integrity of the stadium. The consulting engineers reported that the section was structurally sound. However, an informal inspection by graduate students enrolled in a NDT wood course revealed that the structural members in the horseshoe section were badly decayed and probably would not be able to carry the load from the anticipated crowd. Further evaluation using stress wave equipment (Fig. 16) showed that speed-of-sound transmission was significantly lower in decayed members than in sound wood. Subsequent probing of those areas indicated that the decay was so extensive that only a thin shell of sound wood remained. These results led to the dismantling of the horseshoe section of the stadium. The decay of the timbers was so advanced that when the stress-skin effect of the seating was removed, the substructure collapsed under its own weight.

School Gymnasium

Another structure evaluated with stress wave NDT techniques was a school gymnasium, constructed with laminated barrel arches (Hoyle and Pellerin 1978). These laminated arches were the main support structure for the gymnasium (Fig. 17). Each arch end was exposed to the weather and rested in a metal stirrup fastened to a concrete pier foundation. These conditions and the heavy nonbreathing paint that was used on the exposed portions of the arches created an environment that would support the growth of decay fungi. Cracking and peeling of paint were the first indications that decay was present in the arch ends. When the condition of the gymnasium was realized by school personnel, the problem was one of determining where decay was present and where the wood was sound and did not require replacement. It was not necessary to pinpoint the decayed areas with great precision but to establish how far in from the arch ends that the decay had progressed. The repair procedure was then to replace those ends of the arches with structurally sound material.

The method of inspection was the same as described for the football stadium. To insure that the stress wave travel times were measured in straight lines through individual laminates, a paper, on the third arch from the near end of the gymnasium, containing a grid of 1.5 in. (38 mm) squares, was fastened to each side of the arch and used as a map for taking stress wave time measurements (Fig. 18). The recorded times were then used to determine the extent of the decay (Fig. 19).

Piers

Stress wave techniques were also used to inspect the structural integrity of several piers. Currently limited to inspection of structural components that are above the water line, stress wave techniques were used to inspect a Seattle, Washington, pier that is owned and operated by the U.S. Coast Guard. The pier is constructed of large wood beams and stringers supported on wood piling. Although details of the inspection are not published, NDT techniques similar to those described previously were used.

Bridges

A report by Hoyle and Rutherford (1987) describes the evaluation of wood bridges for the Washington State Department of Transportation using speed-of-sound transmission as an index of deterioration. Previously described stress wave NDT techniques were used. About twelve bridges were evaluated and only one revealed signs of decay. Similarly, Aggour and others (1986) used ultrasonic techniques to evaluate the residual compression strength of timber bridge piles. Relationships between speed-of-sound transmission and residual compressive strength showed excellent correlation.

TRESTLE

TRESTLE was constructed between July 1976 and February 1979 and is one of the largest known glue-laminated structures in the world. It is located at Kirtland Air Force Base, New Mexico. TRESTLE was built as a test stand for aircraft that weigh 550,000 lb (250,000 kg). It has a 50- by 394-ft (15- by 120-m) access ramp and a 200- by 200-ft (61- by 61-m) test platform, and the top surface is 118 ft (36 m) above the ground (Fig. 20).

In the early 1980s, the U.S. Air Force wanted to test aircraft that were considerably heavier than had previously been tested, so they requested a structural evaluation of TRESTLE. One evaluation method relied upon speed-of-sound transmission measurements. Figure 21 shows one stress wave technique that was used. Measurements were taken both longitudinally and transversely to the length of the laminated beams. Neal (1985) and Browne and Kuchar (1985) reported that a total of 484 glulam members (representing approximately 5 percent of the structural members) were evaluated. They concluded that the structural framework of TRESTLE had not measurably degraded, but the exposed deck system was significantly degraded.

Barn Structure

Stress wave techniques were also used to evaluate the wood members of a barn, constructed in 1925 for the College of Agriculture, Washington State University, Pullman, WA (Lanius and others 1981). The structure evaluated was primarily used as an animal shelter on the ground floor and for hay storage on the second floor. The inspection was confined to the nominal 2- by 12-in. (standard 38- by 286-mm) floor joists in the south bay of the barn where hay storage was believed to be the primary use. Speed-of-sound propagation parallel to the grain was measured on 50 percent of the members of the structure. These values were then related to an allowable extreme fiber stress in bending and used to judge remaining strength.

Water Cooling Towers

Stewart and others (1986) used stress wave techniques to evaluate the wood members of several water cooling towers. Using the instrumentation illustrated in Figure 22, approximately 7,700 4-ft- (1.2-m-) long nominal 2- by 4-in. (standard 38- by 89-mm) redwood columns were evaluated. Using the information obtained from a correlation between stress wave parameters and column strength of 74 test specimens and that obtained from the in-place evaluation, individual column strengths were predicted. Columns not meeting desired reliability limits were identified for replacement. This effort resulted in salvaging a substantial portion of the columns that would have otherwise required replacement.

Wood Utility Poles

Anthony and Bodig (1989) reported on the use of sonic stress wave spectral analysis techniques that they had developed and used for inspection of wood structures. Their equipment was designed on the concept that stress waves propagate at different speeds and attenuate differently at various frequencies in wood-based products. Anthony and Bodig collected a time record of a wave propagating through a member, converted it to a frequency spectrum, and then correlated various characteristics to strength using multiple regression analysis techniques (Fig. 23).

Dunlop (1983) utilized an electronic system (Fig. 24), sweeping through a selected range of excitation frequencies, to develop an acoustic signature of a pole. Resonant frequencies were examined for use as NDT parameters.

Other NDT Techniques

Simple mechanical tests are frequently used for in-service inspection of wood members in structures. For example, sounding-, pick-, or probing-type tests are used by inspectors of wood structures to indicate the condition of a structural member. The underlying premise for the use of such tests is that degraded wood is relatively soft and will have a low resistance to probe penetration.

A quantitative test based on the same underlying premise was developed by Talbot (1982). His test differed from the probing-type test in that instead of evaluating probe penetration resistance, Talbot examined withdrawal resistance of a threaded probe, similar to a wood screw, inserted into a member.

Talbot believed that a correlative relationship between withdrawal resistance and residual strength should exist and would be relatively easy to implement. To determine if such a relationship existed, he conducted an experiment using several small Douglas-fir beams that were in various stages of degradation as a result of exposure to decay fungi. Prior to testing to failure in bending, probe withdrawal resistance was measured at the neutral axis of the beams. Bending strength and corresponding probe resistance values were then compared. Talbot's results revealed that a relationship does exist (Fig. 25). He used this test in conjunction with stress wave techniques to assess the extent of damage to the solid-sawn timbers of Washington State University's football stadium.

Concluding Remarks and Future Research Directions

Considerable effort has been devoted to developing NDT techniques for assessing the performance of wood structural members. This report reviewed literature pertaining to NDT of wood, with an emphasis on techniques used for in-place assessment. Based on our review, we conclude the following:

1. A fundamental hypothesis for establishing relationships between NDT parameters and performance of wood members has been established and verified using a wide range of wood-based materials and a variety of NDT techniques.

2. Laboratory investigations on validity of the fundamental hypothesis for establishing predictive relationships for biologically degraded wood, as is sometimes found in structures, have been limited in regards to both the NDT techniques employed and the biological agents of deterioration studied.

3. In-place assessment efforts have focused primarily on adaptations of stress wave NDT techniques. These techniques have shown considerable promise, are relatively easy to use, and have low equipment costs.

Future in-place assessment NDT research should focus on furthering the application of stress wave techniques. Stress wave NDT techniques have been extensively investigated under laboratory conditions and used by inspection professionals on a limited basis. However, many questions remain unanswered regarding the effectiveness of stress wave NDT techniques to evaluate members in complicated structures. No published work documents how wave behavior is affected by the varied boundary conditions found in wood structures. In addition, little information has been published on the relationship between excitation system characteristics and wave behavior. Research efforts in these two areas would advance state-of-the-art inspection techniques considerably.

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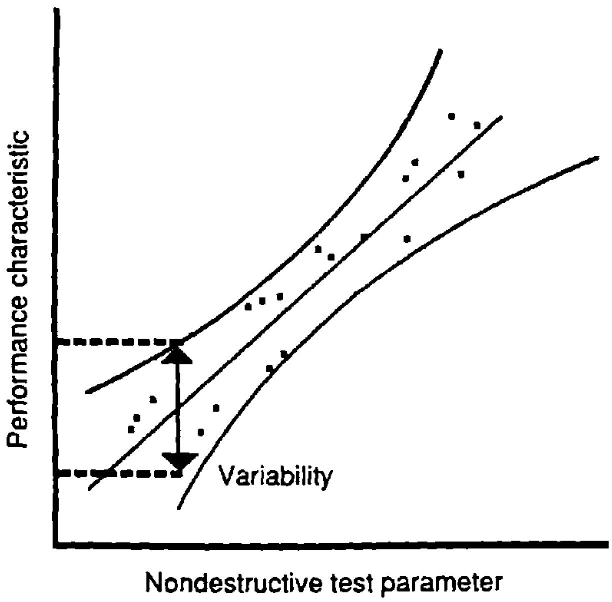

Figure 1—Typical relationship between nondestructive testing parameter and performance.

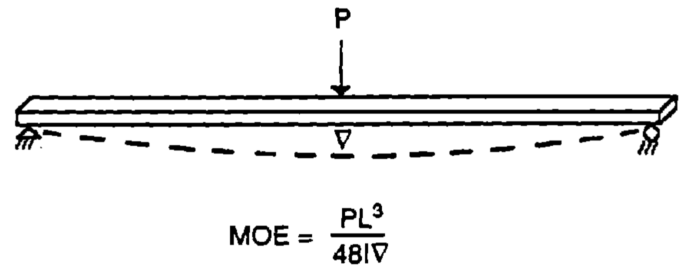

Figure 2—A simply supported beam loaded at its midspan and the mathematical equation relating modulus of elasticity to load and deflection.

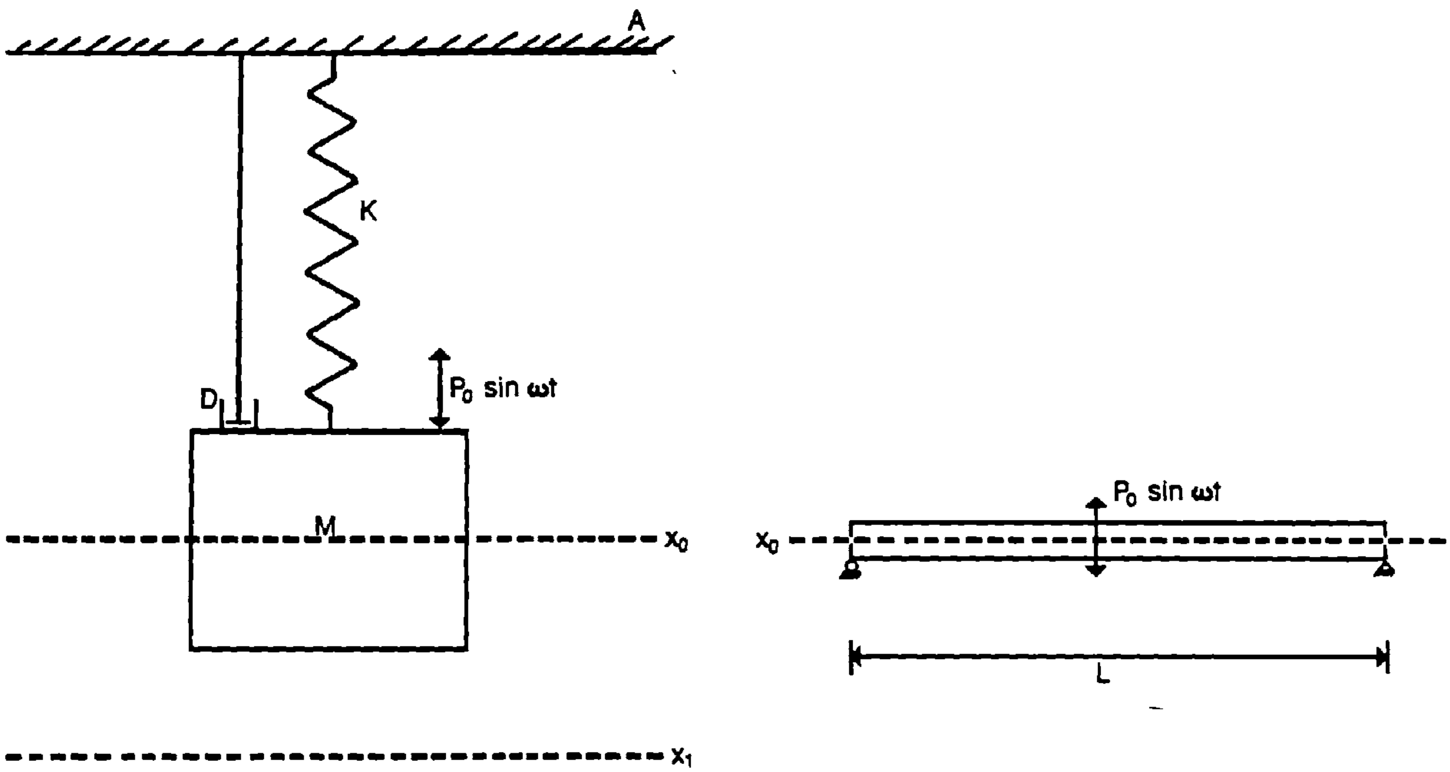

Figure 3—Mass-spring dashpot vibration model (left) and transversely vibrating beam (right).

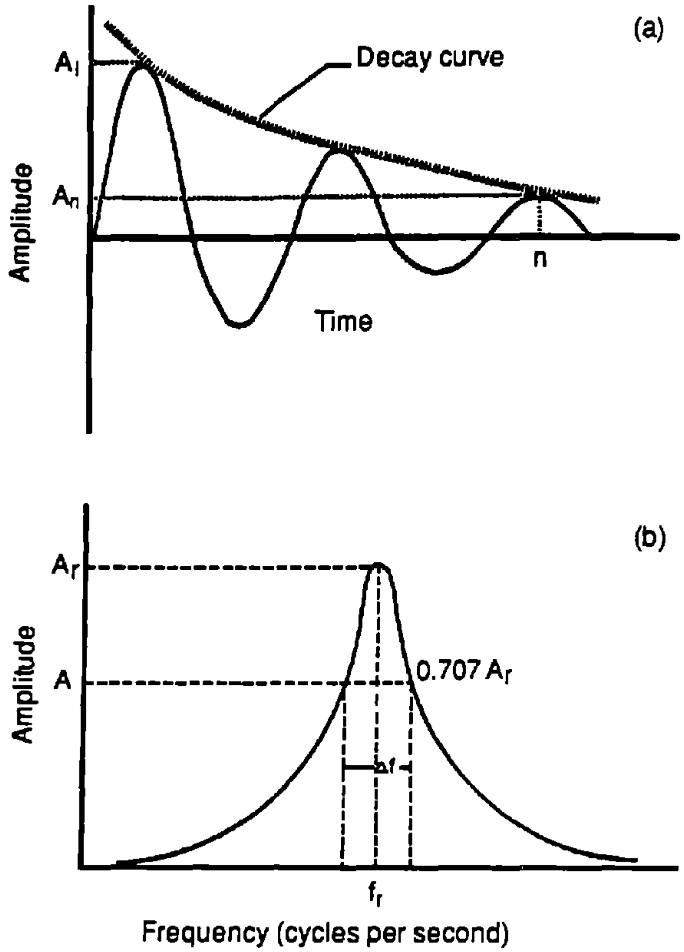

Figure 4—Free vibration of a beam: (a) damped sine wave, (b) frequency response curve.

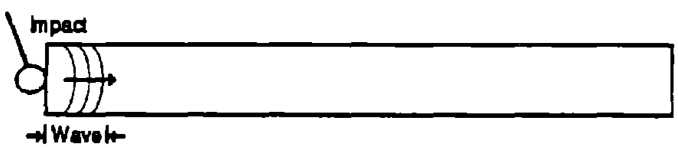

Figure 5—Viscoelastic bar of length L subjected to an impact.

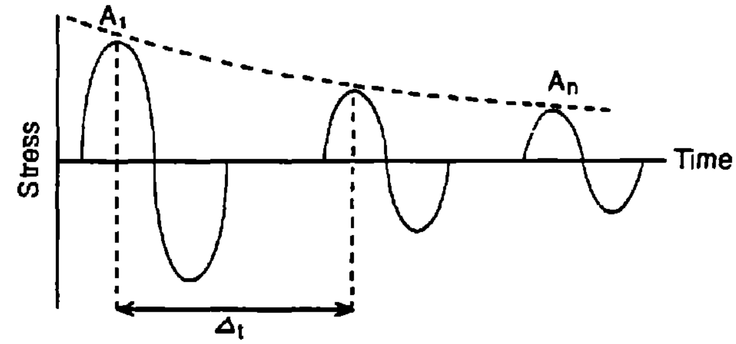

Figure 6—Theoretical response of the end of a viscoelastic bar in response to a propagating stress wave.

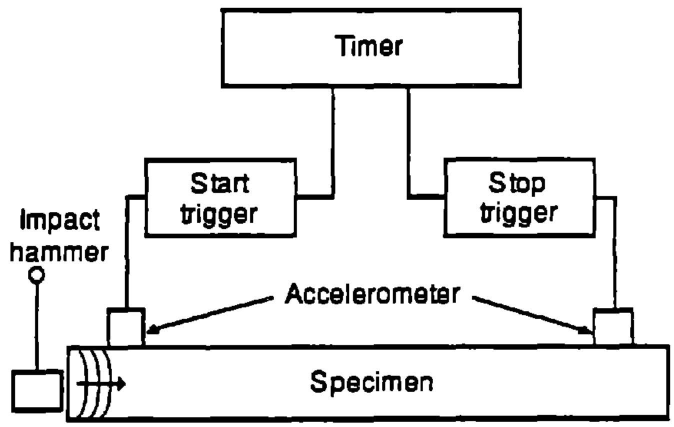

Figure 7—Technique utilized to measure impact-induced stress wave propagation speed in various wood products.

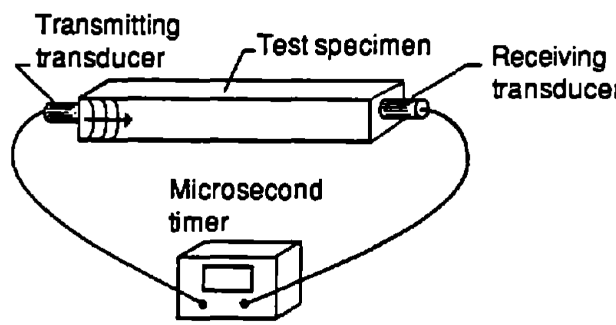

Figure 8—Ultrasonic measurement system used to measure speed-of-sound transmission in various wood products.

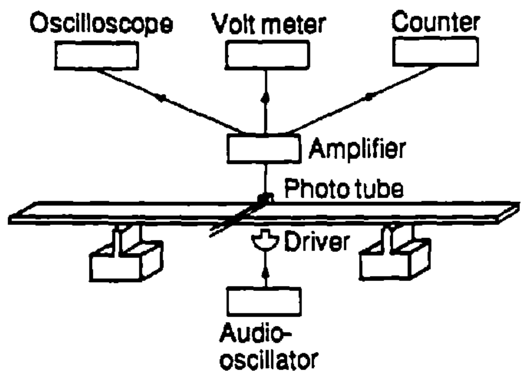

Figure 9—Experimental setup utilized to measure the response of wood beams to forced transverse vibration.

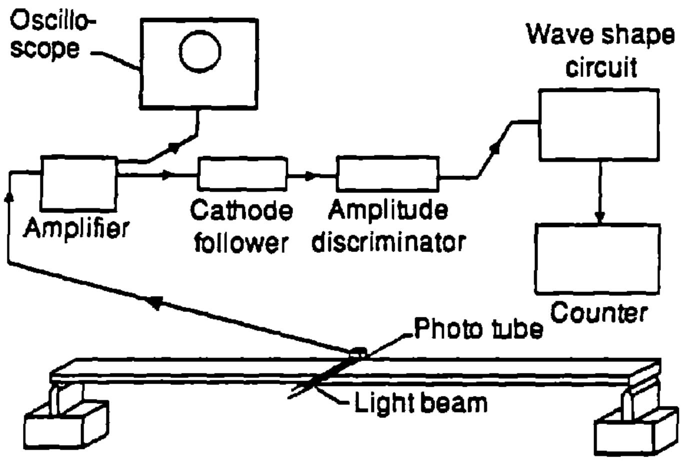

Figure 10—Apparatus used to examine free transverse vibration characteristics of lumber specimens (Pellerin 1965a,b).

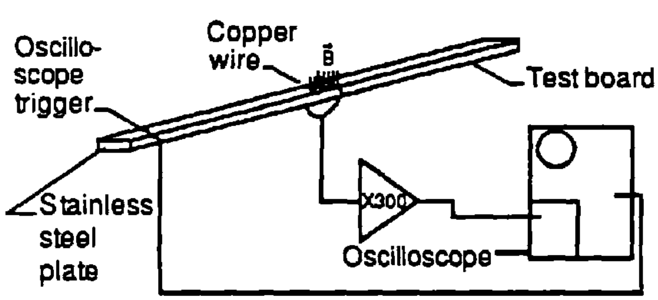

Figure 11—Instrumentation developed to observe stress wave behavior in lumber (Kaiserlik and Pellerin 1977).

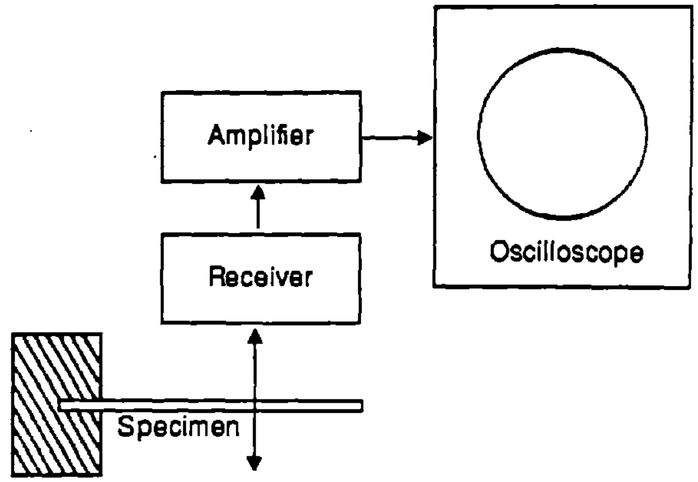

Figure 12—Experimental setup developed to observe free vibration response of decayed specimens.

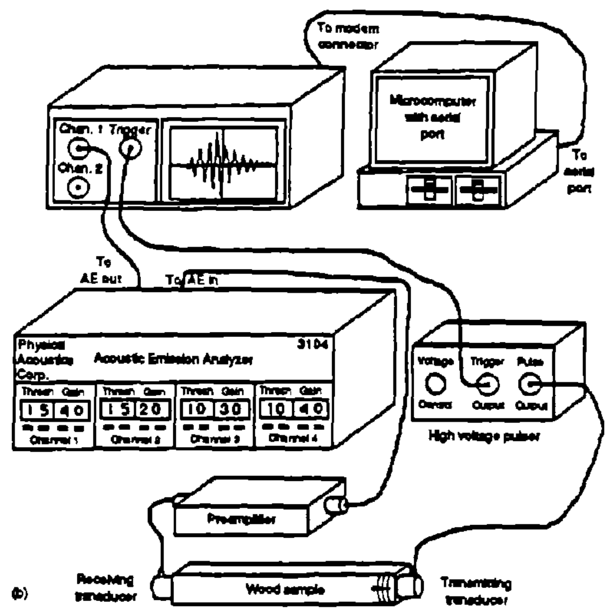

Figure 13—Acousto-ultrasonic equipment (Patton-Mallory and De Groot 1989).

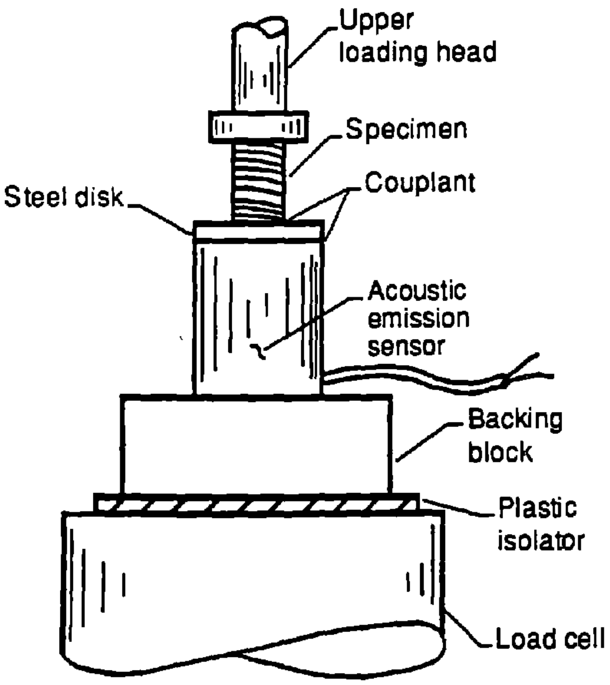

Figure 14—Experimental setup to monitor acoustic emissions from decayed specimens subjected to a compressive force.

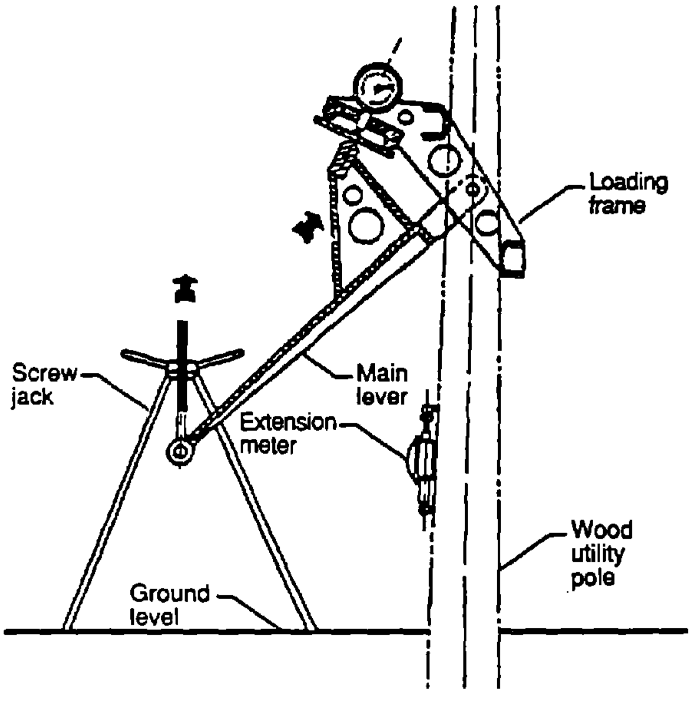

Figure 15—Setup developed to evaluate poles.

Figure 16—Stress wave equipment used to evaluate university football stadium.

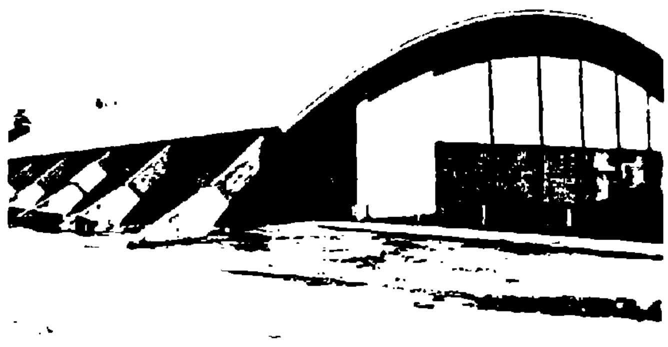

Figure 17—School gymnasium evaluated by Hoyle and Pellerin (1978).

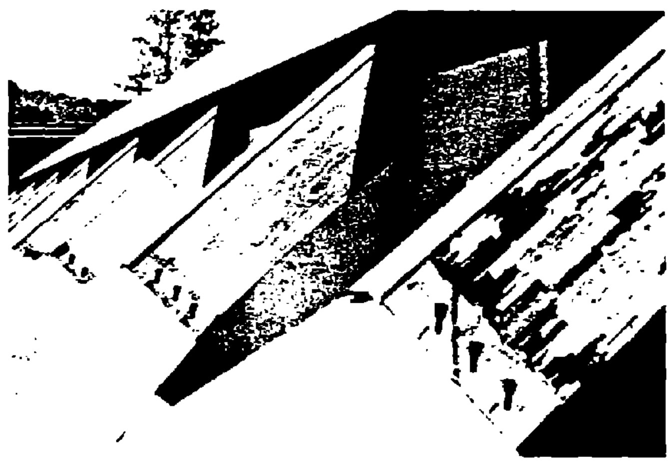

Figure 18—Third barrel arch contains map for stress wave reading.

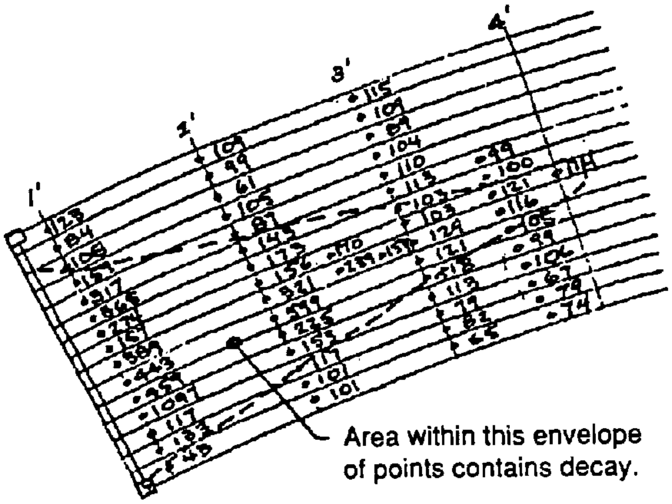

Figure 19—Inspection diagram showing stress wave travel time (μs).

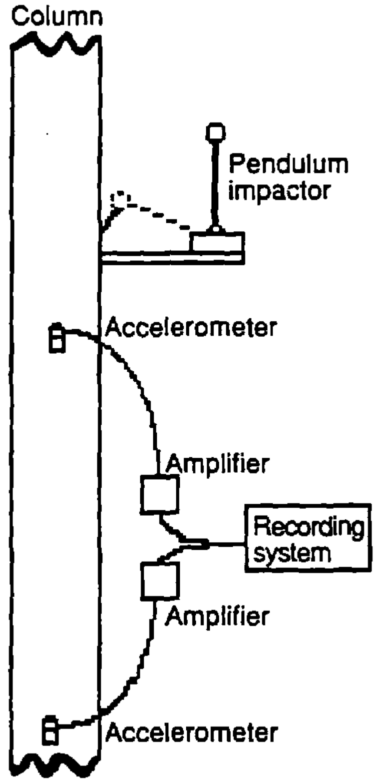

Figure 22—Instrumentation utilized to test wood members in water cooling tower.

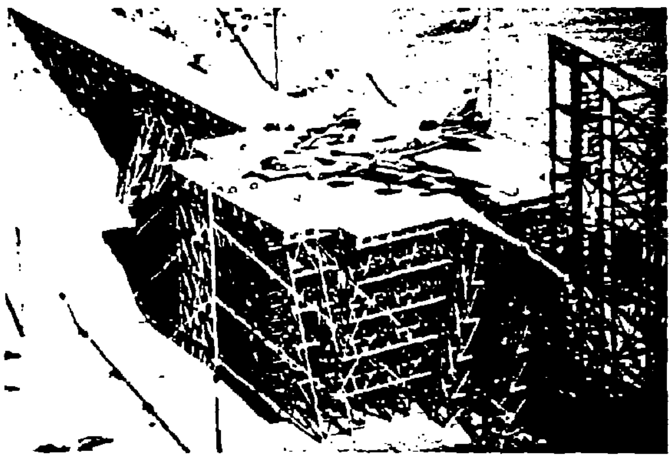

Figure 20—TRESTLE test stand for aircraft.

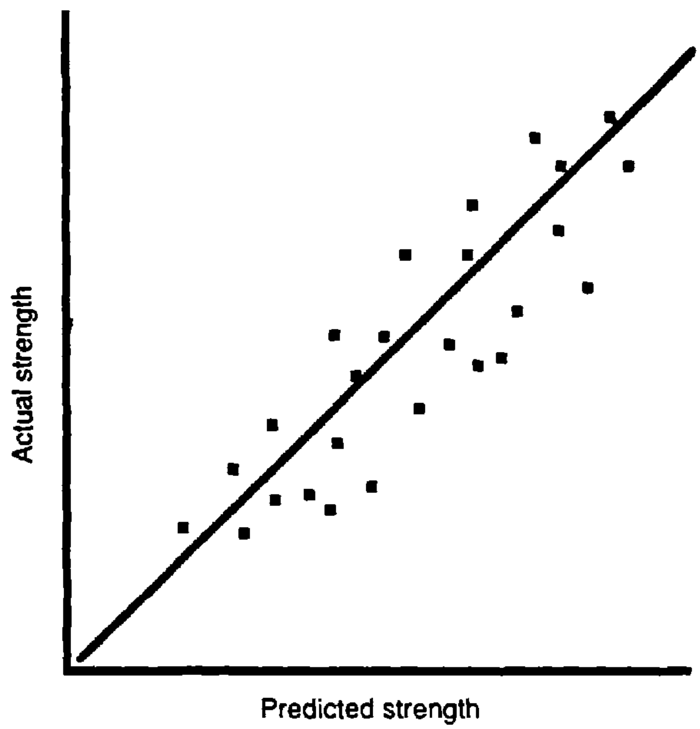

Figure 23—Relationship between predicted and actual strength of utility poles.

Figure 21—Stress wave evaluation of wood members of TRESTLE.

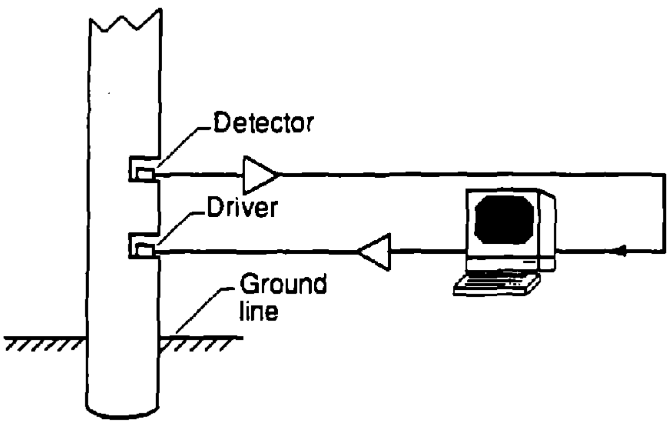

Figure 24—Electronic system to analyze poles.

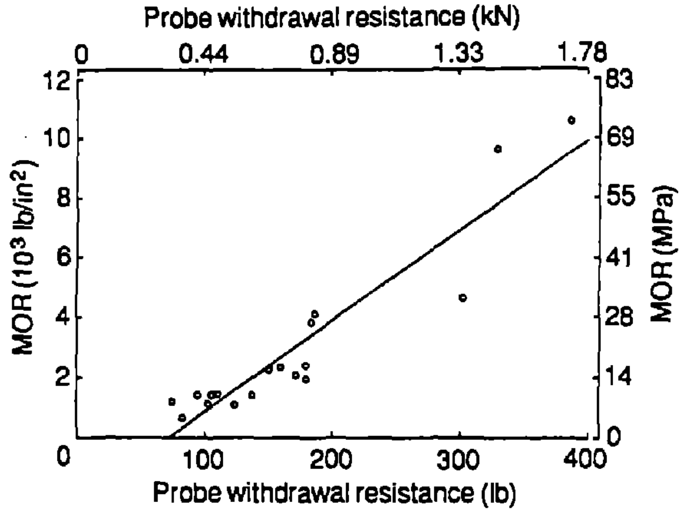

Figure 25—Relationship between probe withdrawal resistance and residual bending strength of Douglas-fir specimens.

Table 1—Research summary on the correlation between stress wave modulus of elasticity values obtained from time-of-flight-type measurements and static modulus of elasticity of various wood materials

Reference	Material	Static loading mode	Correlation coefficient, r
Bell and others (1954)	Clear wood	Compression	0.98
		Bending	0.98
Galligan and Courteau (1965)	Lumber	Bending	0.96
Koch and Woodson (1968)	Veneer	Tension	0.96 – 0.94
Porter and others (1972)	Lumber	Bending	0.90 – 0.92
Pellerin and Galligan (1973)	Lumber	Bending	0.96
	Veneer	Tension	0.96
McAlister (1976)	Veneer	Tension	0.99
Gerhards (1982)	Knotty lumber	Bending	0.87
	Clear lumber	Bending	0.95

Table 2—Research summary on the correlation between modulus of elasticity (tested flatwise) and flatwise bending strength of softwood dimension lumber

Reference	Species	Nominal moisture content (percent)	Grade[a]	Nominal width (in.)[b]	Growth location	Correlation coefficient, r
Hoyle (1961b)	Douglas-fir	12	SS,C,U	4,6,10	Western Oregon, Washington	0.79
					Idaho, Washington	0.72
	Western hemlock	12	SS,C,U	4,6,10	Western Oregon, Washington	0.74
	Western larch	12	SS,C,U	4,6,8	Idaho, Washington	0.70
Hoyle (1962)	Grand fir	12	C,S,U	8	Idaho	0.72
Hofstrand and Howe (1963)	Grand fir	12	C,S	4,6,8	Idaho	0.75
Pellerin (1963b)	Douglas-fir	12	Combination of visual grades	4,8	Idaho	0.76
Hoyle (1964)	Southern Pine	12	1D,1,2D,2,3	4,6,8	Southeastern United States	0.76
Kramer (1964)	Southern Pine	12	1D,2,3	4,6,10	Southeastern United States	0.88
Johnson (1965)	Douglas-fir	10	SS,C,U	6	Western Oregon, Washington	0.85
	Western Hemlock	10	SS,C,U	6	Western Oregon, Washington	0.86

[a]Grades are by regional rules in use at time of research. Western Products Association and West Coast Lumber Inspection Bureau Grades: SS = Select Structural, C = Construction, S = Standard, U = Utility. Western Wood Products Association grades: 1, 2, 3. Southern Pine Inspection Bureau Grades: 1D = No. 1 Dense, 1 = No. 1, 2D = No. 2 Dense, 2 = No. 2, 3 = No. 3.
[b]1 in. = 25.4 mm.

Table 3—Research summary on the correlation between modulus of elasticity (tested flatwise and on edge) and edgewise bending strength of softwood dimension lumber

Reference	Species	Nominal moisture content (percent)	Grade[a]	Nominal width (in.)[b]	Growth location	Correlation coefficient, r
Hoerber (1962)	Douglas fir	12	SS,C,U	4,6,8	Idaho, Eastern Washington	0.65
Hoyle (1962)	Grand fir	12	C,S,U,SS	8	Idaho	0.59 - 0.70
Hoyle (1964)	Southern Pine	12	1D,1,2D,2,3	4,6,8	Southeastern United States	0.57
Sunley and Hudson (1964)	Norway spruce and Scots pine (pooled)	—	—	4,7	Great Britain	0.68
Corder (1965)	Douglas-fir	12	SS,C,S	4,6,10	Inland Northwestern, United States	0.64
Johnson (1965)	Douglas-fir	10	SS,C,U	6	Western Oregon, Washington	0.80 - 0.87
	Western Hemlock	10	SS,C,U	6		0.84
Littleford (1965)	Douglas-fir	10	—	6	British Columbia, Canada	0.74
	Western Hemlock	12	—	6		0.70 - 0.77
	Noble fir	12	—	6		0.66
	Western white spruce	12	—	6		0.79
	Lodgepole pine	17	—	6		0.80
Miller (1965)	White spruce	12	—	6	Eastern Canada	0.78 - 0.84
	Jack pine	12	—	6		0.69 - 0.73
Doyle and Markwardt (1966)	Southern Pine	12	1D,1,2D,2,3	4,6, 8,10	Southeastern United States	—
Hoyle (1968)	Southern Pine	12	1D,1,2D,2,3	4,6,8	Southeastern United States	0.67

[a]Grades are by regional rules in use at time of research. Western Products Association and West Coast Lumber Inspection Bureau Grades: SS = Select Structural, C = Construction, S = Standard, U = Utility.
Western Wood Products Association grades: 1, 2, 3. Southern Pine Inspection Bureau Grades:
1D = No. 1 Dense, 1 = No. 1, 2D = No. 2 Dense, 2 = No. 2, 3 = No. 3.
[b]1 in. = 25.4 mm.

Table 4—Research summary on the correlation between modulus of elasticity (tested flatwise) and the compressive and tensile strength of softwood dimension lumber.

Strength property	Reference	Species	Nominal moisture content (percent)	Grade[a]	Nominal width (in.)[b]	Growth location	Correlation coefficient, r
Compressive	Hofstrand and Howe (1963)	Grand fir	12	Ungraded	4,8	Idaho	0.84
	Pellerin (1963a)	Douglas-fir	12	SS,S,E	4,8	Idaho	0.78
	Hoyle (1968)	Southern Pine	12	1,2,3	4,8	Southeastern United States	0.67
Tensile	Hoyle (1968)	Douglas fir	13	1.0,1.4,1.8,2,2	4,8	Idaho	0.74
		White fir	14			Idaho	0.75
		Western Hemlock	15			Western Oregon, Washington	0.81

[a]Grades are by regional rules in use at time of research. Western Products Association and West Coast Lumber Inspection Bureau Grades: SS = Select Structural, S = Standard, E = Economy.
Western Wood Products Association grades: 1, 2, 3. Machine Stress Grades: 1.0, 1.4, 1.8, 2.2.
[b]1 in. = 25.4 mm.

Table 5—Summary of results that verify the fundamental hypothesis that used transverse vibration and stress wave nondestructive testing (NDT) techniques on clear wood and lumber products[a]

Reference	NDT technique	Material	NDT parameters measured	Static test	Reported properties	Comparison of NDT parameters and static properties (correlation coefficient, r, unless noted)
Jayne (1959)[b]	Forced transverse vibration	Small, clear sitka spruce specimens	Resonant frequency, E_d, Q	Bending	E_{sB}, MOR	E_{sB} and E_d — ±100,000 lb/in^2 MOR and E_d — ±1,000 lb/in^2 MOR amd E_d — ±1,000 lb/in^2 MOR and density/Q — ±1,000 lb/in^2 MOR and E_d/δ — ±900 lb/in^2
Pellerin (1965a)	Free transverse vibration	Douglas-fir glue-lam	Natural frequency, E_d, δ	Bending	E_{sB}, MOR	Predicted relative strength of three glue-laminated members.
Pellerin (1965b)	Free transverse vibration	Inland Douglas-fir dimension lumber	Natural frequency, E_d, δ	Bending	E_{sB}, MOR	E_{sB} and E_d — 0.98 MOR and E_d — 0.67–0.93 MOR and $1/\delta$ — 0.46–0.88 MOR and E_d/δ — 0.68–0.92
O'Halloran (1969)	Free transverse vibration	Lodgepole pine dimension lumber	Natural frequency, E_d, δ	Bending	E_{sB}, MOR	E_{sB} and E_d — 0.98 MOR and E_d — 0.89 MOR and $1/\delta$ — 0.82 MOR and E_d/δ — 0.91
Kaiserlik and Pellerin (1977)	Longitudinal stress wave	Douglas-fir boards	C, E_d, δ	Tension	UTS	UTS and E_d — 0.84 UTS and combination of E_d and δ — 0.90

[a] C = Speed of sound.
δ = Logarithmic decrement.
E_d = Dynamic modulus of elasticity obtained from either transverse vibration or stress wave measurements.
E_{sB} = Modulus of elasticity obtained from static bending test.
MOE = Modulus of elasticity.
MOR = Modulus of rupture.
Q = Sharpness of resonance.
UTS = Ultimate tensile stress.
1 lb/in^2 = 6.9 × 10^3 Pa.
[b] Correlation coefficients were not reported by Jayne. However, he did report 95 percent confidence intervals.

Table 6—Summary of results that verify the fundamental hypothesis using wood-based composites[a]

Reference	NDT technique	Material	NDT parameters measured	Static test	Reported properties	Comparison of NDT parameters and static properties (correlation coefficient, r, unless noted)
Suddarth (1965)	Forced transverse vibration	Laminated wood (missile noise fairing)	E_d, δ			Mapped out debonded or poorly bonded areas.
Pellerin and Morschauser (1974)	Longitudinal stress wave	Underlayment particleboard	C	Bending	E_{sB}, MOR	E_{sB} and C^2 — 0.93–0.95 MOR and C^2 — 0.87–0.93
Ross (1984), Ross and Pellerin (1988)	Longitudinal stress wave	Underlayment and industrial particleboard, structural panel products	C, E_d, δ	Tension	E_{sT}, UTS	E_{sT} and C^2 — 0.98 E_{sT} and E_d — 0.98 UTS and C^2 — 0.91 UTS and E_d — 0.93 UTS and $1/\delta$ — 0.63 UTS and combination of E_d, $1/\delta$ — 0.95
				Bending	E_{sB}, MOR	E_{sB} and C^2 — 0.97 E_{sB} and E_d — 0.96 MOR and C^2 — 0.93 MOR and E_d — 0.92 MOR and $1/\delta$ — 0.70 MOR and combination of E_d, $1/\delta$ — 0.97
				Internal bond	IB	IB and combination — 0.79
Fagan and Bodig (1985)	Longitudinal stress wave	Wide range of wood composites	C	Bending	MOR	Simulated and actual MOR distributions were similar.
Vogt (1985)	Longitudinal stress wave	Medium-density fiberboard	C, E_d, δ	Tension	E_{sT}, UTS	E_{sT} and C^2 — 0.90 E_{sT} and E_d — 0.88 UTS and C^2 — 0.81 UTS and E_d — 0.88 Combination — 0.88
				Bending	E_{sB}, MOR	E_{sB} and C^2 — 0.76 E_{sB} and E_d — 0.72 MOR and C^2 — 0.96 MOR and C^2 — 0.92 Combination — 0.97
Vogt (1986)	Stress wave (through transmission)	Underlayment and industrial particleboard, structural panel products	C_t, E_{dt}	Internal bond	IB	IB and C_t^2 — 0.70–0.72 IB and E_{dt} — 0.80–0.99

[a] C = Speed of sound.
C_t = Speed-of-sound transmission through thickness.
δ = Logarithmic decrement.
E_d = Dynamic modulus of elasticity obtained from either transverse vibration or stress wave measurements.
E_{dt} = Dynamic modulus of elasticity, through the thickness orientation.
E_{sB} = Modulus of elasticity obtained from a static bending test.
E_{sT} = Modulus of elasticity obtained from a static tension test.
MOR = Modulus of rupture.
UTS = Ultimate tensile stress.

Table 7—Research summary of correlation between nondestructive testing (NDT) parameters and properties of degraded wood[a]

Reference	NDT technique	Material	Degradation agent	NDT parameters measured	Static test	Reported properties	Comparison of NDT parameters and static properties (correlation coefficient, r, unless noted)
Wang and others (1970)	Free transverse vibration (cantilever bending)	Small, clear eastern white pine sapwood specimens	Brown-rot fungi (*Poria placenta* Murr.)	Natural frequency	None		Significant loss in frequency as early as 7 days after inoculation
Chudnoff and others (1984)	Longitudinal stress wave (parallel to grain)	Decayed and sound mine props; 26 species or species groupings	—	E_d	Compression parallel to grain	E_c, UCS	E_c and E_d — 0.84–0.97 (all species combined, hardwoods, maple, and oaks). E_c and E_d — 0.73–0.81 (all species combined, southern pines, lodgepole pine). UCS and E_d — 0.85–0.95 (all species combined, hardwoods, maple, and oaks).
Pellerin and others (1985)	Longitudinal stress wave (parallel to grain)	Small, clear southern yellow pine specimens	Brown-rot fungi (*Gloeophyllum trabeum*)	C, E_d	Compression parallel to grain	UCS	UCS and C: 0.47 (controls) 0.73 (exposed) 0.80 (control and exposed) UCS and E_d: 0.86 (controls) 0.86-0.89 (exposed) 0.94 (control and exposed)
			Termites (subterranean)	C, E_d			UCS and C: 0.65 (controls) 0.21 (exposed) 0.28 (control and exposed) UCS and E_d: 0.90 (controls) 0.79 (exposed) 0.80 (control and exposed)

Beall and Wilcox (1986)	Acoustic	Small, clear white fir specimens	Brown-rot fungi (*Poria placenta*)	AE	Compression	Stress at various levels	AE events were very sensitive to degree of mass loss and stress level.
Rutherford and others (1987a,b)	Longitudinal stress wave (perpendicular to grain)	Small, clear Douglas-fir specimens	Brown-rot fungi (*Gloeophyllum trabeum*)	C, E_d	Compression perpendicular to grain	E_c, UCS	E_c and C — 0.91 E_c and E_d — 0.94 UCS and C — 0.67–0.70 UCS and E_d — 0.79 UCS and MOE — 0.80
Patton-Mallory and De Groot (1989)	Longitudinal stress wave	Small, clear southern yellow pine specimens	Brown-rot fungi (*Gloeophyllum trabeum*)	C, root mean square voltage frequency content of received signal	Bending	Maximum moment, alkali solubility	C decreased in a linear fashion with increasing decay degradation. Signal strength decreased with increasing decay degradation. High-frequency components of signal were attenuated with very early stages of decay degradation.

[a]AE = Acoustic emission events.
C = Speed of sound.
E_c = Modulus of elasticity obtained from a static compression test.
E_d = Dynamic modulus of elasticity obtained from either transverse vibration or stress wave measurements.
MOE = Modulas of elasticity.
MOR = Modulus of rupture.
UCS = Ultimate compressive stress.

Table 8—Research summary of nondestructive testing (NDT) concepts for in-place evaluation of wood structures[a]

Reference	NDT technique	Type of structure	Location	Material	NDT parameters measured	Analysis performed—conclusions
Lee (1965)	Longitudinal stress wave	Eighteenth century mansion roof	United Kindgom	Solid-sawn timber	C	Developed empirical relationship between speed-of-sound transmission and residual strength.
Hoyle and Pellerin (1978)	Longitudinal stress wave (perpendicular to grain)	School building	Idaho	Curved glulam arches (span 120 ft, rise 33 ft)	C	Detected decay in exposed ends of arches. Mapped out areas of decay.
Lanius and others (1981)	Longitudinal stress wave	Barn	Washington	2- by 12-in. joists	C, E_d	Estimated residual strength of members.
Dunlop (1983)	Acoustic resonance	Wood poles	Australia	Wood utility poles	Resonant frequencies	Test diagnosed large percentage of poles in sample set correctly.
Browne and Kuchar (1985)	Longitudinal stress wave	Dielectric support stand for testing large aircraft in a simulated flight situation	New Mexico	Glulam, structural timbers	C, E_d	MOE determined, strength properties inferred.
Neal (1985)	Longitudinal stress wave (parallel and perpendicular to grain)	Large military test stand (TRESTLE)	New Mexico	Glulam	E_d	Structural framework was not degraded; exposed deck system was degraded.
		Small military test stand	New Mexico	Glulam	E_d	Structural framework and decks were degraded.
		Large military test stand	Arizona	Glulam, solid sawn timber	E_d	Accessible structural degradation had not occurred.
Aggour and others (1986)	Longitudinal stress wave (perdendicular to grain)	Bridge piling	Maryland	Piling	C, density	Correlation of density and C to compressive strength of pile ($r = 0.98$).

Abbott and Elcock (1987)	Full-size static MOE test	Wood poles	United Kingdom	Wood utility poles	Bending MOE	Correlative relationship between MOE and residual strength of poles ($r = 0.68$).
Hoyle and Rutherford (1987)	Longitudinal stress wave (parallel and perpendicular to grain)	Timber bridges	Northwestern United States	Solid sawn timber	C, E_d	Revealed signs of decay in 1 of 12 bridges; reevaluation every 3 years.
Murphy and others (1987)	Vibration	Wood poles	Western Canada	Wood utility poles (Douglas-fir cedar)	Resonant frequencies	Comparison to pole stiffness ($r = 0.82$).
Anthony and Bodig (1989)	Stress wave	Wood cooling tower, poles	Texas, Western United States	Solid sawn timber, poles	C, δ, phase shifts	Determined rate of strength degradataion.
Pellerin (1989)	Longitudinal stress wave	University football stadium	Washington	Solid sawn timber	C	Found severe decay degradation; structure was dismantled. substructure collapsed under its own weight.
		Piers	Washington	Large wood beam, stringers supported by wood pilings	C	Replaced structural members containing decay.

[a]C = Speed of sound.
δ = Logarithmic decrement.
E_d = Dynamic modulus of elasticity obtained from either transverse vibration or stress wave measurements.
MOE = Modulus of elasticity.
r = correlation coefficient.
1 ft = 0.3 m, 1 in. = 25.4 mm.

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COMPOSITE BRIDGES AND NDE APPLICATIONS

Joseph M. Plecknik
Civil Engineering Dept.
California State University, Long Beach
Long Beach, CA 90840

Oscar Henriquez
Graduate Research Assistant
California State University, Long Beach
Long Beach, CA 90840

1. INTRODUCTION

The search for solutions to problems regarding bridge structures has turned the attention of researchers to possible application of new materials such as composites, mainly the so called fiber-reinforced plastics (FRP). Corrosion is one of the most important factors which cause significant damage to present bridge structures. Composite materials, when properly designed, can resist almost any environmental attack, and therefore are good candidates to be used in bridges. Use of composite materials, for their light-weight properties, can also lead to rehabilitation and upgrading of existing structurally deficient bridges, thus allowing that bridges be used for loads as originally designed. Of all the structural elements used in bridges, decks are the most feasible to be fabricated with composites. Besides presenting the advantages to be highly corrosion resistance and of light weight, FRP decks also have the advantage of feasibility for modular construction, low impact load factors and possible life-cycle costs. Also cables for suspension bridges are being tested, as well as prestressing tendons which have already been used.

New structural systems made of composite materials are promising for application as structural elements in bridges; however, it is very unlikely that their use be accepted unless techniques are developed for testing and evaluating their in-service performance. Advances in NDE techniques as applied to composite structures have been made, and some of these are feasible to be used on composite bridges, and are currently being used for testing of composite bridge deck models and suspension cables; however, there is still a considerable amount of work to be done in this area so that the techniques can be applied to in-service composite bridges, as it is explained in this paper. Applications of NDE techniques to evaluate in-service performance of civil engineering structures other than bridges are also presented in this paper since currently there is no experience with full-scale composite bridges evaluated by NDE methods.

2. COMPOSITES AND NDE IN CIVIL ENGINEERING STRUCTURES

Composite materials have already been widely used in civil engineering structures. Currently, some of the most common applications of composites as civil engineering structures are underground tanks, storage tanks, stacks, tanker trucks and others. Several NDE techniques are used to evaluate the performance of these structures. The most common type of composites are by far fiber-reinforced plastics (FRP), which are composed of thin fibers bound up in polymer resins. Appropriate fibers are glass, Kevlar and graphite; other fibers with higher strengths usually are too expensive for these applications. For matrices, epoxy or polyester resins are used in most of the composites currently fabricated.

NDE methods have been applied to evaluate the performance of underground and storage FRP tanks. Underground tanks, a very common and successful application of FRP composites, are usually made with chopped glass fibers and used to store highly corrosive chemicals. The most used NDE technique to inspect underground tanks has been ultrasound, which is used to locate and identify defects and damage. A similar application is found in storage tanks, which also can resist the attack of corrosive chemicals. These tanks are usually fabricated by filament winding technique; however, in this case acoustic emission is the NDE most used to locate damage and evaluate the performance of the tank.

Both NDE techniques, ultrasound and acoustic emission, as applied for inspecting underground and storage tanks, have a point or local scope, which means that only a localized area can be evaluated at a time. To be able to apply these methods to inspect a large structure such as a bridge, it would require a great amount of time and/or equipment. Similarly, strain gages are used to monitor strains and stresses on points of the surface of the tanks.

An interesting approach to NDE in composite structures is the use of high modulus carbon fibers within a glass-fiber composite to monitor the performance of a structure on a global scope, as opposite to the point and local scopes. Graphite fibers, unlike the glass fibers, have the property of conducting electricity, and high-modulus graphite fibers have a higher modulus of elasticity and some also have lower tensile strength than those of glass fibers; therefore if in a glass-fiber lamina a carbon fiber is embedded, the latter would fracture first under high strains. The carbon fiber could then serve as a sensor if its electric conductivity or resistance is monitored, and an alarm is triggered when the breakage of the fiber produces a change in those measurements; then, one would have the idea that the level of stresses has reached a point where damage of the composite may be close, and could take some preventive action to avoid failure.

The graphite-fiber sensors NDE technique has been applied to some composite structures. One of those types of structures is high pressure pipes or small tanks, which can take pressures up to 3,000 to 4,000 psi. These pipes are filament wound and graphite coils are introduced with the glass-fiber principal reinforcement during the fabrication process. An identical approach is used in composite tanker trucks which are much larger than the mentioned high pressure pipes, but are also fabricated using the filament winding process. In both cases, the graphite sensors are very useful because they can be oriented in the direction of maximum stresses, and therefore it is very feasible to detect failures which would occur as a result of these stresses. Similarly, this method has also been applied to FRP stacks, which under the effect of wind forces can deflect significantly, thus monitoring its performance is very important, and embedded graphite filaments were used for this purpose. It is important to notice that this method is only a type of alarm that will alert us when the graphite fiber has been subjected to excessive strain; however, other methods may be required to further evaluate the possible damage of the structure.

All the structures which have been described here may be considered two-dimensional surfaces. In order to apply the graphite-fiber sensors NDE method to structures of more complicated geometries, such as those that may be necessary for composite bridges, modifications have to be made, otherwise the scope of the technique would be limited and not global as it may be desired. As we can see now, the size and geometry of a structure to be monitored is an important factor to consider when deciding what NDE methods to use.

3. COMPOSITES IN BRIDGE STRUCTURES

Engineers are finding that FRP composite materials are feasible to be used in bridge structures. Some of the bridge components are more suitable to be fabricated of composites than others, and those are the ones that are being developed, such as decks, cables and prestressing tendons. Also, composites have already been used for retrofitting of bridges components such as columns and girders.

The one structural bridge component that makes more sense to be made of FRP composites is the deck. An all fiber-glass reinforced plastic composite bridge deck with an X-shaped cross section has been under testing. Several cross sections, were considered for analysis and the most effective one in structural and economical terms was chosen for fabrication and testing. A deck such as this one would have several advantages. Light weight properties of composites could be reflected in a reduction of 70 to 80%[1] in the weight of the deck as compared to a reinforced concrete deck, and the total dead load reduction of the bridge superstructure could be about 54%. This weight reduction could allow for an increase in allowable live load, use of additional lanes using the same superstructure, and updated use of structurally deficient bridges. Another very important advantage of this type of deck is their high resistance to corrosive environment, currently a major problem for bridge decks, thus reducing maintenance and reparation costs, and also possibly lowering life-cycle costs. In addition, a system of modular construction is feasible, consequently reducing assembly time and lowering installation costs. Another advantage of FRP decks is that they have similar damping properties than wood, hence reducing impact loads and allowing to neglect this factor for design. On the other hand, FRP decks have some disadvantages: lighter weight could have negative effect on the dynamic response of the structures; FRP composite

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[1]Plecknik, J.M. and Ahmad, S.H., Transfer of Composites Technology to Design and Construction of Bridges, prepared for U.S. DOT, Federal Highway Administration.

materials are affected by fire and elevated temperatures; and a composite bridge structure may have a higher initial cost.

Specimens of FRP bridge decks of different sizes have been tested, and also inspected using several NDE techniques. Static and fatigue tests have been performed to determine the response of specimens under loads equivalent to HS20-44 AASHTO truck and Military load standards. Nondestructive evaluation of the bridge deck, a complex composite structure, becomes difficult when it is considered that different failure modes may be developed either individually or combined; the most typical failure modes are matrix cracking, fiber failure, delamination, corrosion of fibers and/or resin, stress-corrosion, debonding, and matrix plasticization. Those failure modes may be caused by different factors such as fatigue, creep, impact, corrosion, temperature differential, mechanical and environmental abrasion, and ultraviolet rays.

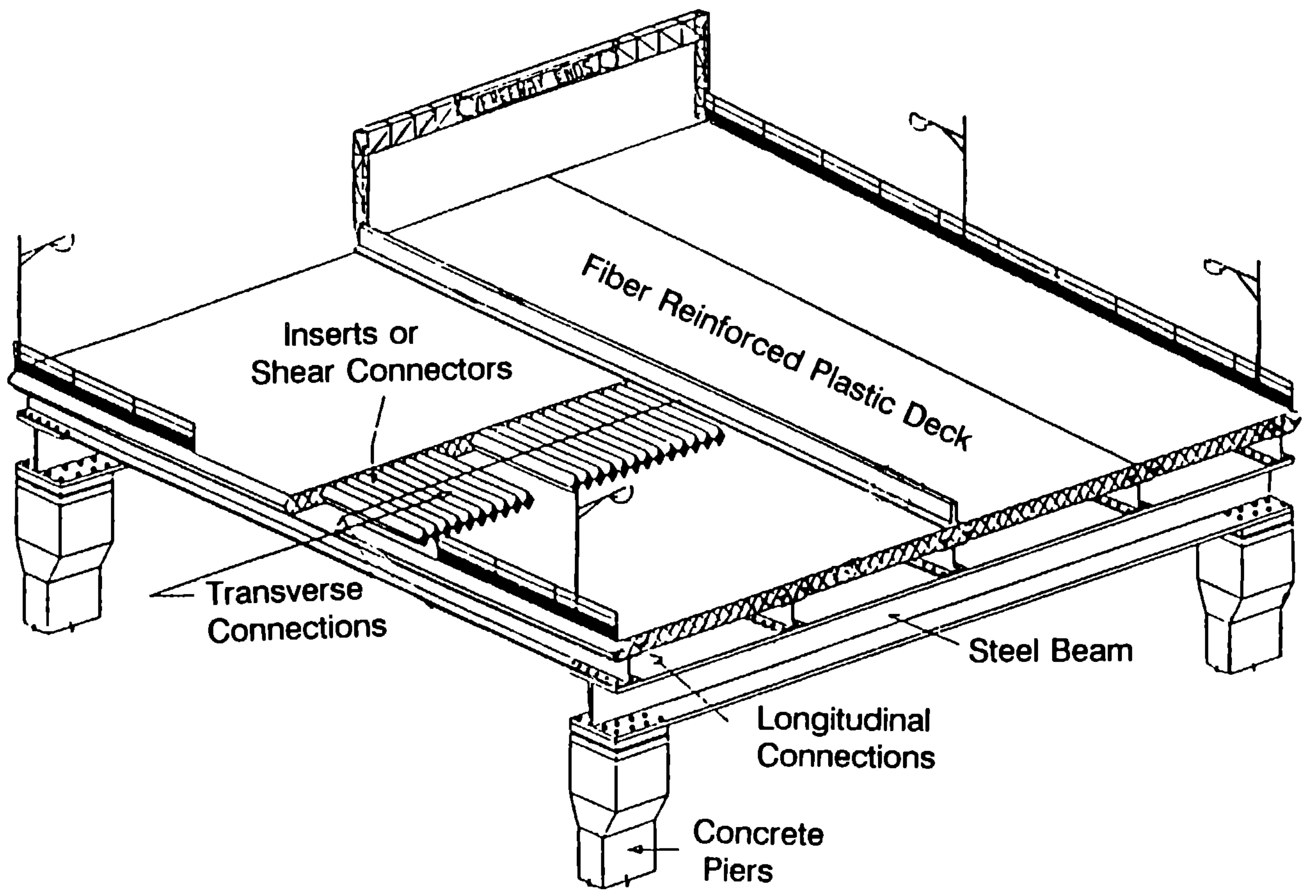
In evaluating deck specimens under fatigue loads, some NDE techniques have given better results than others. On a point and local scope, acoustic emission has been the most useful method to detect damage as it happens; the type of failure may be inferred in some cases from the intensity and amount of acoustic emission. Since different failure modes produce characteristic AE for a given type of material, variations in specific ranges of AE amplitude may be an indication of propagation of some damage. If the AE results show that the failure may be significant, then one has an idea that the load level may produce failure of the specimen, and a reduction of the load may be necessary to continue the fatigue test. Even though this method has produced satisfactory results in testing deck samples, it is unlikely that it will be a technique of global scope when applied to a real bridge deck, which would be much larger than the specimens tested so far, because of the great amount of AE sensors that would be necessary due to the fact that each sensor has only a local scope; this is a result of the acoustic attenuation within the material. Other drawbacks of this method would be that it is not intended for continuous monitoring of the structure, and that extraneous noise has to be eliminated or filtered so that it does not interfere with the results. The three-dimensional geometry of the deck introduces accessibility problems, and that makes also more difficult the evaluation and location of damage.

Other techniques have also been used to evaluate the FRP deck specimens. Ultrasound was one of the methods which was considered a very serious candidate, however the results have not been too promising, mainly due to the fact that it is very slow for evaluating a large structure of complicated geometry as the deck. Application of ultrasound technique may be to inspect a relatively small area, once one has a good idea where the failure may be located, and in that way find out exactly the location and type of damage; however, accessibility may be a difficulty. Another simple but more limited technique is the use of strain gages, which have only a point scope, and therefore would give information of what is happening at one specific point, which may not be sufficient enough to evaluate the structure because stresses in a composite lamina can vary significantly throughout its thickness; however, the information may be useful when it is combined with data from other methods.

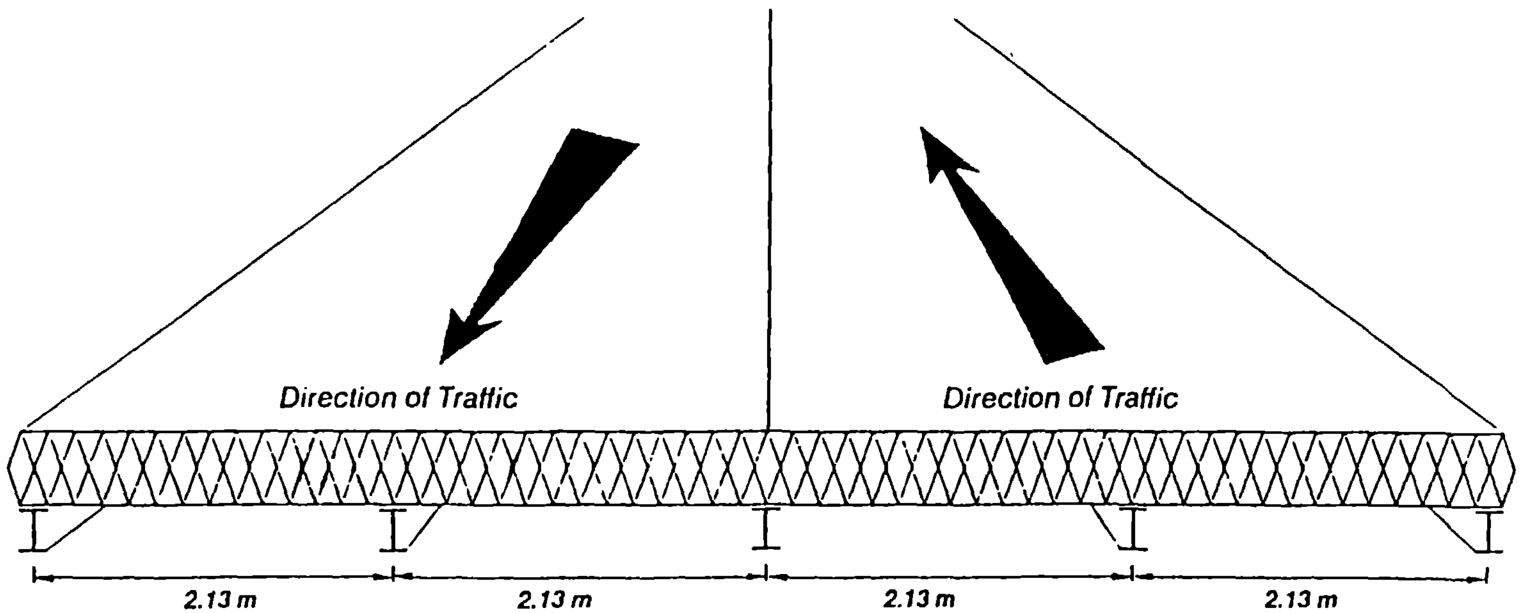
Development of NDE techniques of global scope should be a goal for evaluating FRP bridge decks. As mentioned before, the use of graphite fibers embedded during fabrication process within the glass-fiber composite to continuously monitor failure, is an NDE method which could have a global scope, and therefore its application should be further developed as to be suitable for composite decks.

In addition to decks, composites can be used in other bridge applications. Link-type bridge cables have been manufactured for testing. Main reinforcement of the cables was glass fibers, and some graphite was added to form the anchorage zone; the matrix was polyester resin. Static and fatigue tests were performed, and they showed that failure always occurs at the end or anchorage of the cable. Strain gages were used to monitor the performance of the cables, and the total elongation was also measured. The graphite-fiber sensors technique can be easily applied to the case of cables, which are not a complicated structure and the location of critical areas is very well known. The same concept of cables has been applied to prestressing tendons, where also fiber optics technique has been applied to monitor failure of the tendons. Composites have also been used to retrofit bridge structures; carbon-fiber and glass-fiber reinforced plastics have been used to repair and reinforce bridge members such as columns and beams.

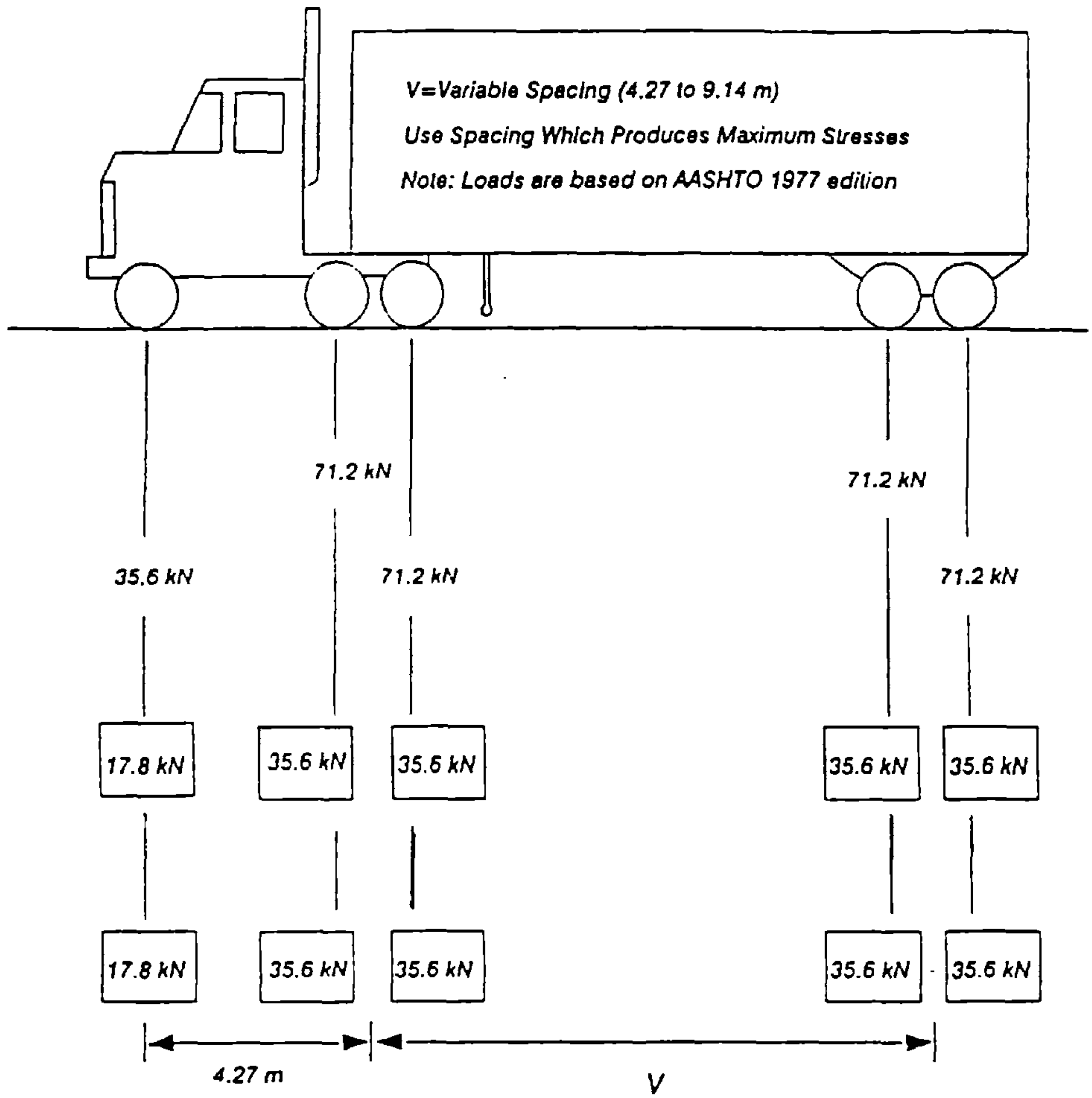
Some bridge components are not likely to be made completely of composite materials. Structural members such as columns and beams require a stiffness which cannot be achieved with the most common composites, therefore their use may be limited in this area. Advanced composite materials, which have better mechanical properties than other composites, are not feasible at this time to be used in bridges for economical factors.



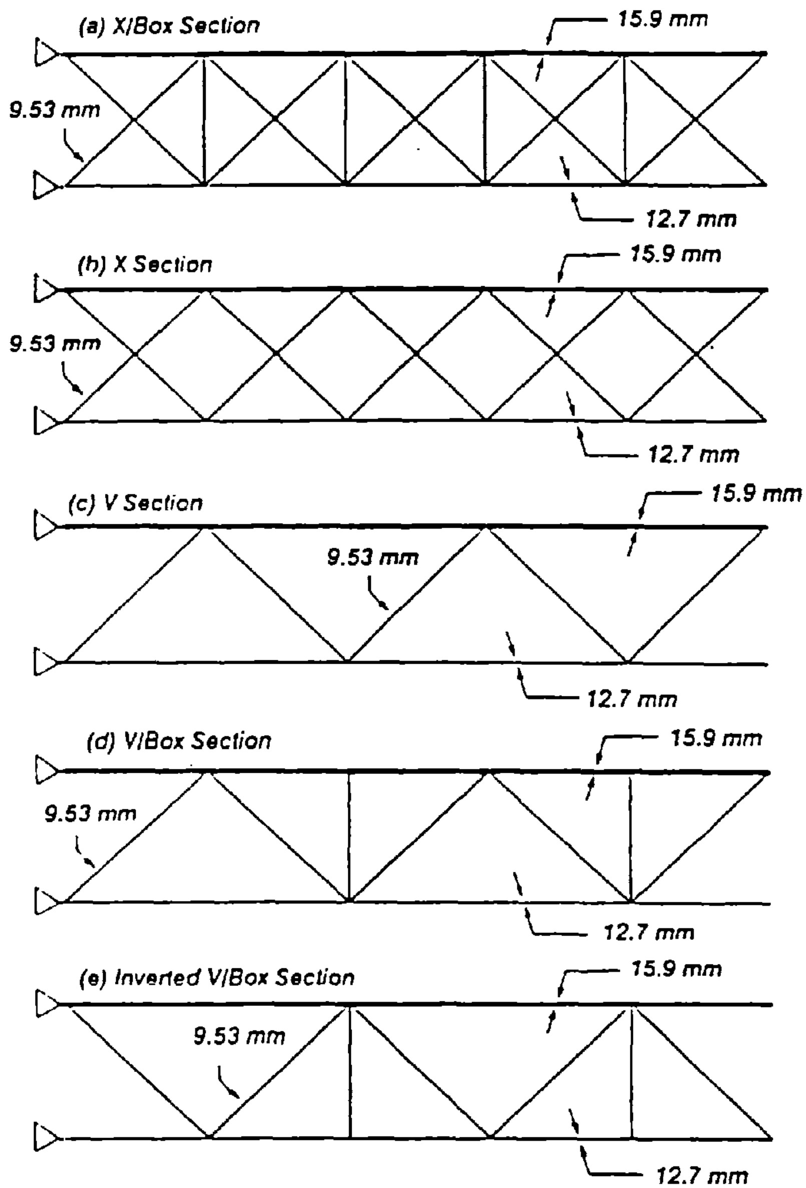
Perspective View of an FRP Bridge



Four-Span Fiber Reinforced Plastic Bridge Deck



AASHTO HS 20-44 Loading Specification

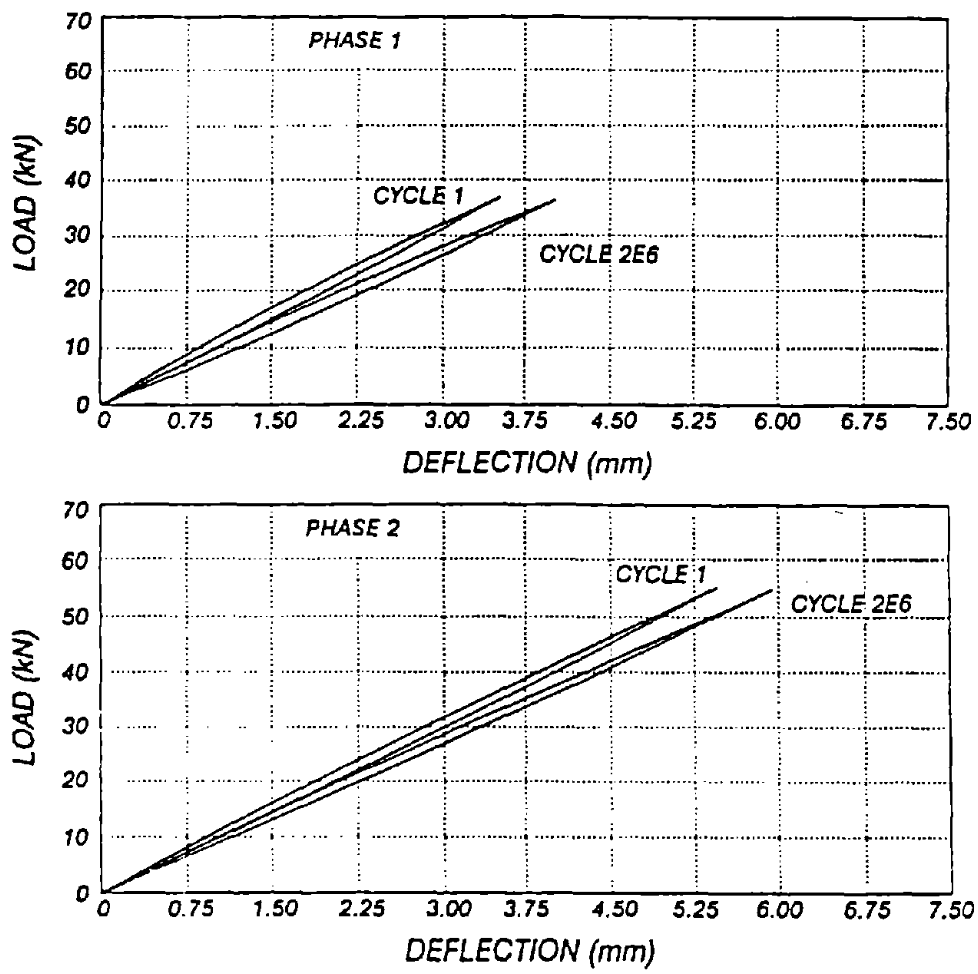


Deck Shapes Considered for Analysis

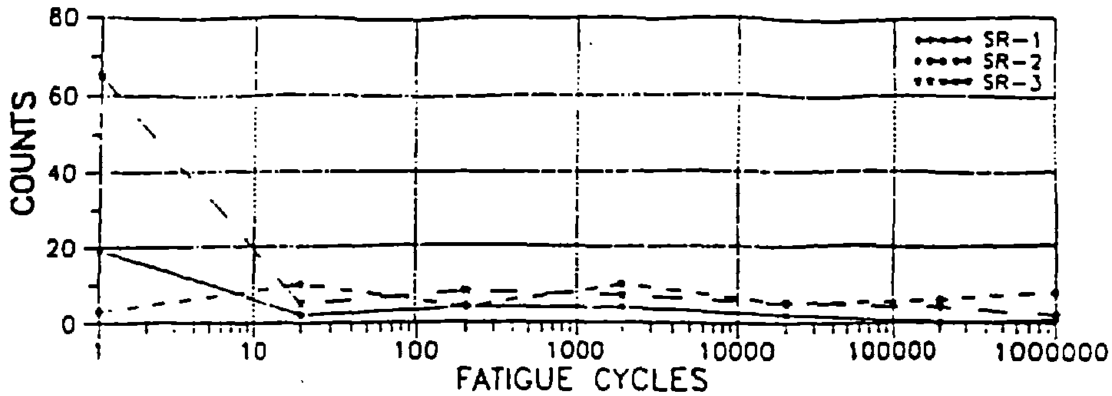
Comparisons of Dead Load of FRP Deck and Superstructure

Bridge Type	Bacule with 1.22 m Stringer Spacing Span = 76 m, Width = 18.8 m				Deck on Steel I-Girder (W27x94) with 2.13 m Girder Spacings. Span = 16.3 m Width = 8.51 m.		Deck on AASHTO Type III Prestressed Girders Spaced at 2.13 m and 16.3 m Span. Width = 8.51 m.	
Deck Type	12.7 cm Steel Grid	15.2 cm Deep X-Shaped FRP With Sand Layer Wearing Surface	12.7 cm Concrete Filled Steel Grid	15.2 cm Deep X-Shaped FRP With Sand Layer Wearing Surface	16.5 cm Thick Concrete with 5.1 cm Wearing Surface	22.9 cm Deep X-Shaped FRP with Sand Layer Wearing Surface	17.8 cm Concrete With 5.1 cm Wearing Surface	22.9 cm Deep X-Shaped FRP With Sand Layer Wearing Surface
Deck Wt. Only (kN)	1380	1160	5650	1160	540	160	580	160
Deck D.L. % Reduction	16.0		80.0		71.0		73.0	
Girder Wt. (kN)	3600				110		690	
Curbs and Railing (kN)	300				170		170	
Future Wearing Surface (kN)	0.0	17.8	0.0	17.8	170	2.2	170	2.2
Details, Stiffeners etc. (kN)	290				35.7		23.8	
Inspection Walkway (kN)	67				None		None	
Total D.L. (kN)	5650	5440	9920	5440	1020	475	1630	1040
Total D.L. % Reduction	4.0		45.0		54.0		36.0	

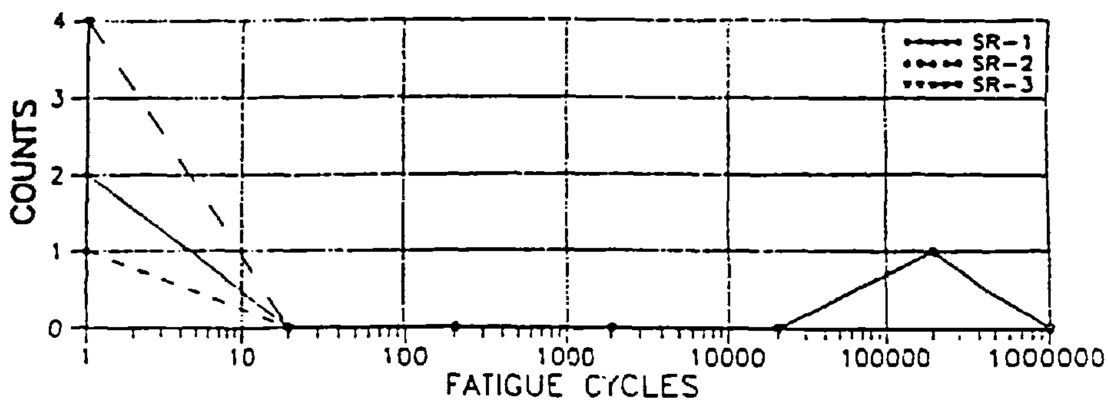
(1 Kip = 4.448 kN, 1 in = 2.54 cm, 1 ft = 0.304 m)



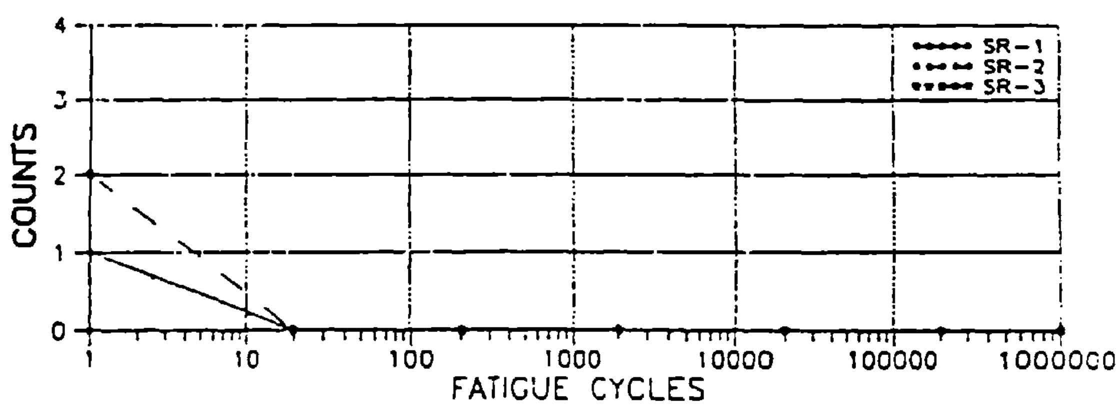
Load vs. Deflection for FRP Bridge Deck



(a) Counts (40 to 60 dB) vs. Fatigue

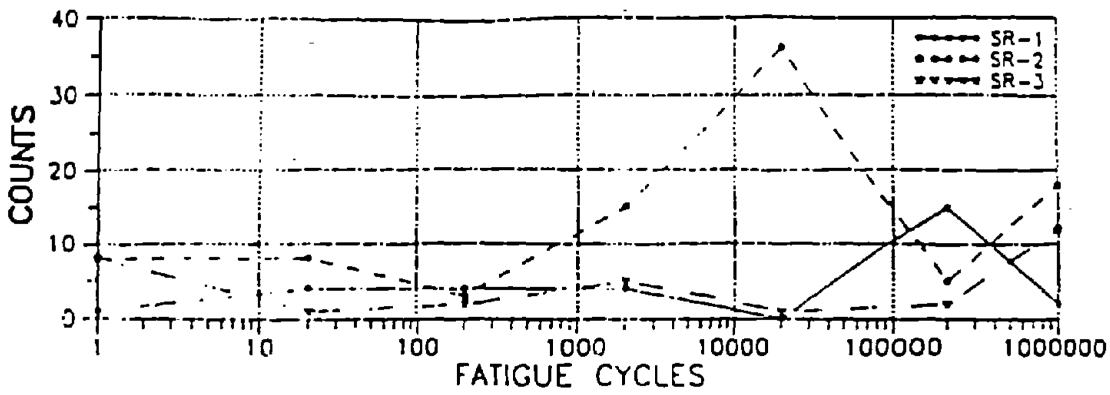


(b) Counts (60 to 70 dB) vs. Fatigue

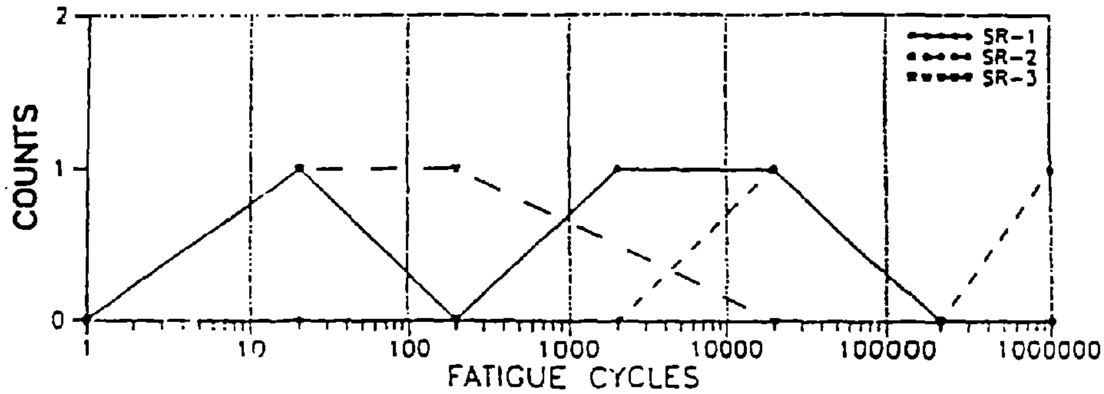


(c) Counts (70 to 100 dB) vs. Fatigue

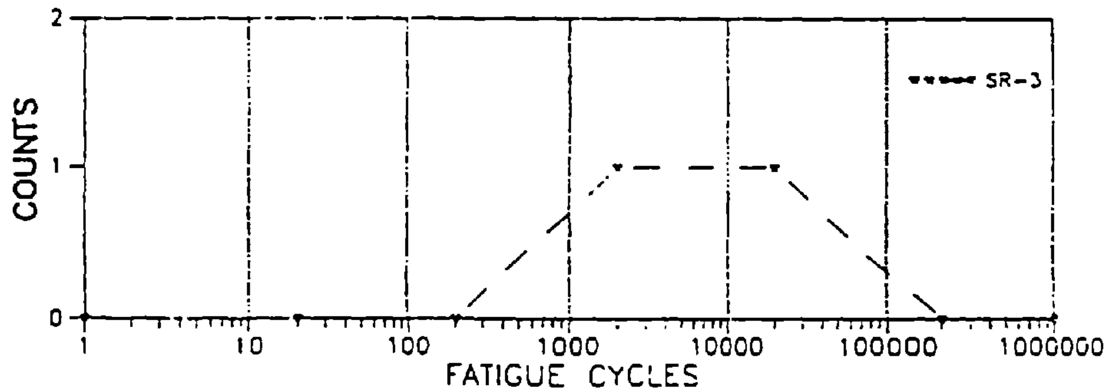
Counts vs. Fatigue for 1D-1 Phase 1



(a) Counts (40 to 60 dB) vs. Fatigue

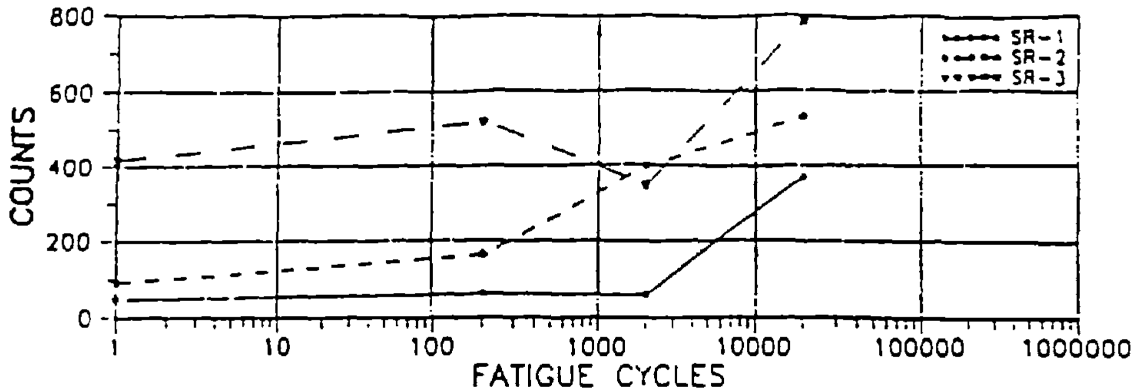


(b) Counts (60 to 70 dB) vs. Fatigue

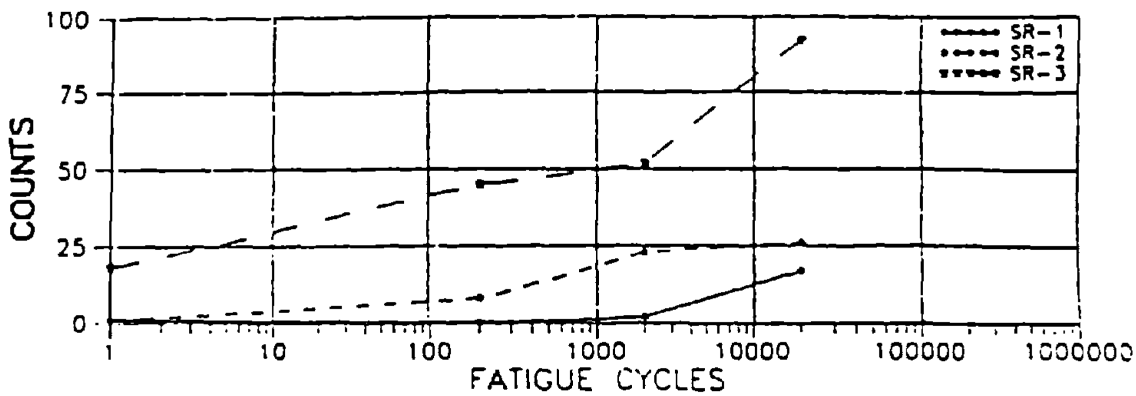


(c) Counts (70 to 100 dB) vs. Fatigue

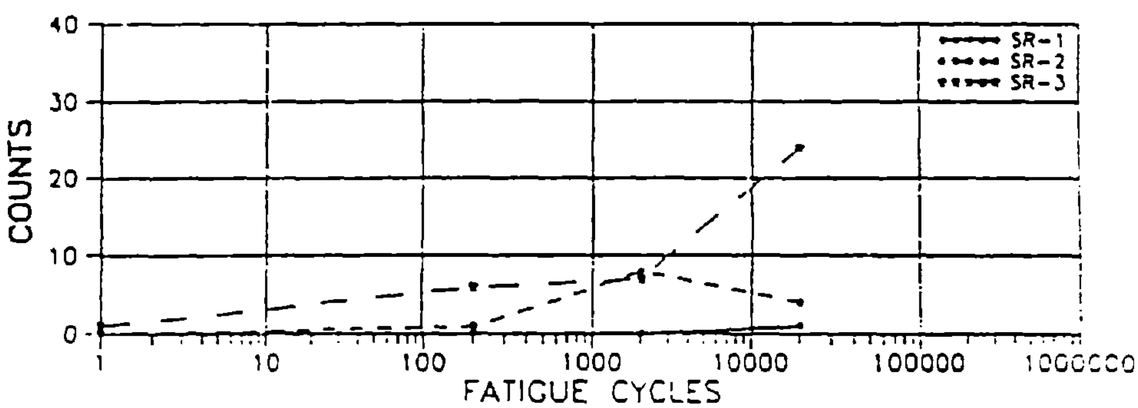
Counts vs. Fatigue for 1D-1 Phase 2



(a) Counts (40 to 60 dB) vs. Fatigue



(b) Counts (60 to 70 dB) vs. Fatigue



(c) Counts (70 to 100 dB) vs. Fatigue

Counts vs. Fatigue for 1D-1 Phase 3

SUMMARY OF FATIGUE PARAMETERS

Specimen ID	Phase	Maximum Load (lbs)	R	Cycles	Damage
ID-1	1	4000	.75	1E6	NONE
	2	4000	.50	1E6	A
	3	5000	.20	20K	B,C,D
ID-3	1	4500	.22	2E6	A
	2	5000	.20	1E6	C
2D-1	1	4500	.25	1E6	A
	2	5500	.25	127K	C,D
2D-2	1	4000	.25	2E6	NONE
	2	4750	.25	2E6	A
3D-1	1	4500	.25	2E6	NONE
	2	5000	.25	2E6	A
3D-2	1	5000	.25	2E6	A
15D-1	1	8500	.25	2E6	A
	2	12500	.25	200K	A,B
15D-2	1	8500	.25	2E6	A
	2	12500	.25	2E6	A
15D-3	1	12500	.25	1E6	A
	2	11250	.25	2E6	A
	3	12500	.25	1E6	A

A = Debonding between the epoxy paste and filament wound parts.
B = Audible resin/delamination cracking.
C = Excessive visual delamination.
D = Complete failure of hand lay-up laminate.

TYPICAL FAILURE MODES FOR COMPOSITE BRIDGES

Matrix cracking

Fiber failure

Delamination

Corrosion of fibers/resin

Stress-corrosion

Debonding

Matrix plasticization

DAMAGE CLASSIFICATION

Purpose: Determine load changes in next phase of fatigue loading

Failure

Critical

Subcritical	Reduce load in next phase
Significant	Load maintained constant in next phase
Insignificant	Increase load in next phase

DAMAGE MODES FOR COMPOSITE BRIDGE DECKS

1. Fatigue - truck wheel loads
2. Creep - long term dead loads
3. Impact - truck wheel loads
4. Corrosion - moisture, salt, transported chemicals
5. Temperature Differential - expansion, contraction

        Mechanical - car & truck tires
        /
6. Abrasion
        \
        Environmental - sun, rain, wind, freeze-thaw

7. UV

LAMINATE DAMAGE LOCATION

Internal

Micro-Voids
Macro-Voids
Interlaminar Fiber or Resin Cracking

External

Surface Fiber or Resin Cracking
Surface Deterioration

STRUCTURAL CLASSIFICATION OF NDE TECHNIQUES

GLOBAL
(Total Stiffness or deflection)

LOCAL
(AE, Thermography, Radiography)

POINT
(US, Strain Gages)

POSSIBLE NDT TECHNIQUES FOR COMPOSITE BRIDGES

TECHNIQUE	SCOPE
Acoustic Emission	Local, Point
Radiography	Point
Thermography	Local, Point
Fiber Optics	Global, Local
Holography	Local, Point
Photo Elasticity - Coatings	Local, Point
Electrical Resistance Tech (Strain Gages, Graphite Coils)	Global, Local, Point
Visual Inspection	Global, Local, Point
Eddy Currents	Local, Point
Ultrasound > 20 kHz	Local, Point
Liquid Penetration Systems	Local, Point
Magnetic Particle Techniques	Global, Local
Sonic Techniques (100 Hz-20 kHz)	Global, Local

THERMOGRAPHY

Uniform heat flux applied to one side of surface.

Voids or defects within structure will modify the heat flow patterns resulting in non-uniform heat flux on other sides of structure.

NOTES:
1. Expensive
2. Complex
3. Not for large scale field applications

RADIOGRAPHY AND MICROWAVE TECHNIQUES

Penetrating radiation through composite results in various absorption of radiation. Where void present, less radiation absorbed, which is detected by a film.

1. X-ray example
2. Good for location in composites of
   a) large voids
   b) delaminations
   c) cracks
3. Too expensive, especially for large field applications

ULTRASOUND TECHNIQUES

High frequency (over 20 kHz) sound waves sent through thickness which are affected by voids, damage or interfaces.

TYPES

1. Liquid Immersion
2. Water Jet
3. C-scan
4. Through Transmission
5. Pulse Echo - most feasible for composite bridge applications

SCOPE - Local, Point

NOTES:
1. Consistent for metals, short fiber (hand lay-up) composites, and advanced composites.
2. Inconsistent (due to extensive voids) for commercial filament winding composites.
3. 1/4 (Art) + 1/4 (Science) + 1/2 (Experience)
4. Applicable when loads not applied.
5. Good for laboratory, too localized for field application to complex composite bridges.

ACOUSTIC EMISSIONS

Records and evaluates energy waves (sound waves from damage)

SCOPE: - Local, Point

NOTES:
1. Can detect internal damage
2. Affected by externally produced vibrations (lighting, vehicle noise, wheel impact, etc.)
3. Rapid attenuation in composites
4. 1/3 (art) + 1/3 (science) + 1/3 (experience)
5. Excellent for fatigue studies
6. Good for laboratory, too localized for field application to complex composite bridge decks
7. Applicable only when loads applied
8. Future advances may lead to global application

FIBER OPTICS

Passage of light through thin glass-like fibers is affected by stress patterns.

SCOPE: - Global, Local

NOTES:
1. Most useful for tensile stress fields
2. Post-Tensional Cables in German Bridge
3. Excellent as NDE for composite cables
4. Can be embedded into laminate
5. Can find stress magnitude and location

MAGNETIC PARTICLE MATRICES

BASIC IDEA: Mix magnetic dust with matrix. Electromagnetic fields change with stress variations.

SCOPE: Global, Local

MATERIAL: Resin + Magnetic Dust + Non-Magnetic Fibers (0.25%)

ADVANTAGES: Inexpensive, Large Scale Applications

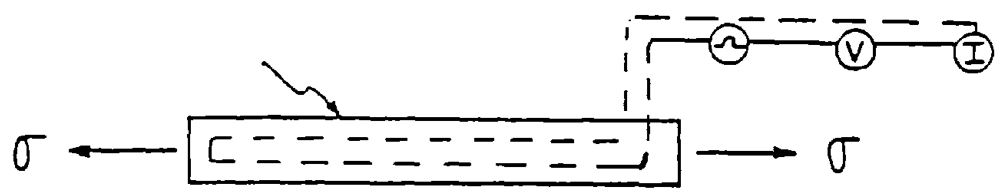
DISADVANTAGES: Two-Dimensional

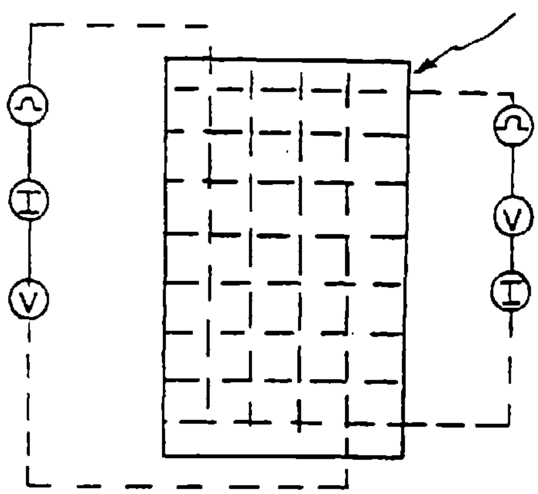
CURRENT APPLICATIONS:

1. Centercore
2. Tank Precipitators

GRAPHITE COILS

BASIC IDEA: High modulus-low strength filament embedded into laminate or lamina of low modulus-high strength fibers.





SCOPE: Global and Local Applications

NOTES:
1. Inexpensive
2. Placed during fabrication
3. Used for static strength, fatigue, creep
4. Simple and. unaffected by field conditions
5. Limited to tensile stress fields

Properties of Glass Reinforcement Fibers

Property	Glass Type				
	g[a]	S-2[a]	g[b]	R[b]	T[c]
Density, g/cm^3	2.54	2.48	2.60	2.55	--
Tensile strength, psi x 10^3					
at 22°C	500	665	495	640	675
at 370°C	380	645	--	--	--
at 450°C	--	--	260	430	--
Tensile modulus, psi x 10^6					
at 22°C	10.5	12.4	10.6	12.5	12.2
Elongation at break, %	4.8	5.7	--	--	--

[a]Ownes-Corning Fiberglas Company.

[b]Vetrotex Saint-Gobsin.

[c]Nitto Boseki Company, Ltd.

Properties of Commercially Available Carbon Fibers

Property	Magnamite[a] Type AS4	Magnamite[a] Type IM6	Thornel[b] T-40 12K	Thornel[b] 300 (PAM) WYP 15-1/0	Thornel[b] P-100 2K (Pitch)	Torayca[c] T400	Beefight[d] ST	RK Carbon[e] RK35
Tensile strength, psi x 10^3	520	635	820	450	325	470	>500	508
Tensile modulus, psi x 10^6	34	40.4	40-42	33.2	105	34	34	32-35
Strain to failure, %	1.5	1.5	2.0	1.3	0.31	--	1.5	1.48-1.85
Density, g/cm^3	1.80	1.73	1.81	1.77	2.15	1.74	1.77	1.78
Filament diameter,	8	-	6	7	10	7	7	6.8
Carbon content, %	94	94	92	92	99+	--	--	95

[a]Hercules Incorporated.

[b]Amoco Chemicals Company (Union Carbide).

[c]Toray Industries, Inc.

[d]Toho Beslon Company, Ltd.

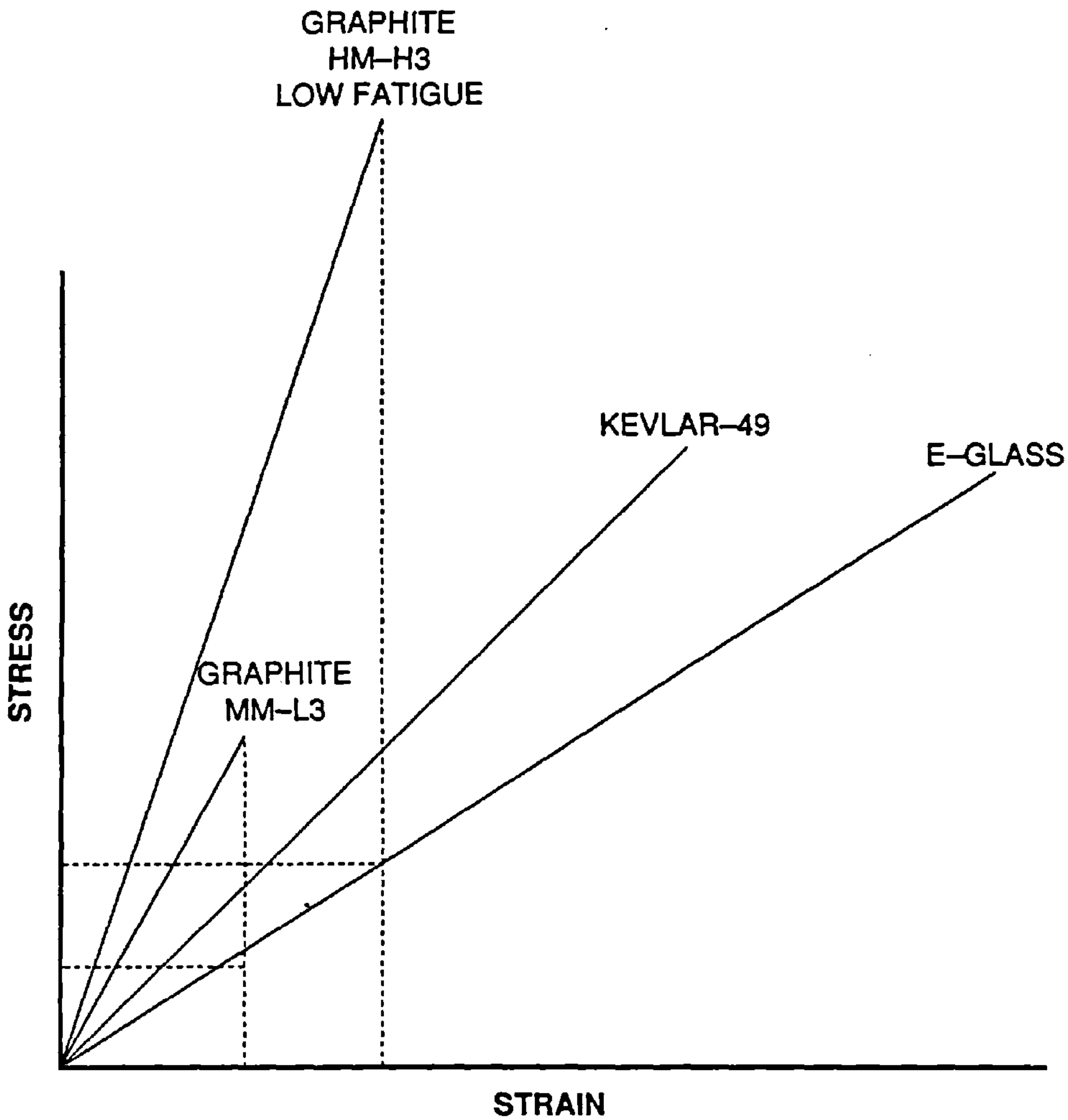
[e]RK Carbon Fibres, Ltd.

Typical Properties and Prices of Reinforcing Fibers for Advanced Composites

Fiber	Density (g/cm^3	Tensile Strength (psi x 10^3)	Tensile Modulus (psi x 10^6)	Price[a] (dollars per pound)
Carbon (PAM-based)				
High strength	1.8	650	40	
High module	1.9	350	75	15 to 80
General purpose	1.8	350-400	30-35	
Glass, E	2.55-2.60	500	10.5	0.70-0.80
Glass, S-2 or R	2.50-2.55	665	12.6	4.50-6.50
Aramid (Kevlar 49)[b]	1.44	400	18-20	15-50
Polyethylene				
(Spectra 900)[c]	0.97	375	17	22
Boron	2.7	500	58	300
Silicon carbide				
(Micolon)[d]	2.55	400	28	400

[a]The price ranges shown cover various product forms. Carbon fiber price depends heavily on filament count. The lowest prices are for 12,000 filaments per strand; the highest are for 1,000 filaments per strand. Prices for specialty grades of carbon fiber can reach as high as $600 per pound.

Sources: [b]Du Pont
[c]Allied-Signal Corporation
[d]Mippon Carbon Company



Fiber Characterization for NDE

THE STATE-OF-THE-ART IN
NONDESTRUCTIVE EVALUATION OF
STEEL BRIDGES

Cecil M. Teller
Southwest Research Institute
San Antonio, TX

1. INTRODUCTION

In 1967, the collapse of the Silver Bridge at Point Pleasant, West Virginia, ushered in the modern era of nondestructive evaluation (NDE) research and development (R&D) for highway bridges. This bridge failure resulted in the loss of almost 50 lives and subsequently was determined to be due to fatigue cracking and brittle fracture of a non-redundant tension chord member (steel eye-bar). Many other examples of even more recent bridge failures could be cited which also resulted in loss of life and property, but suffice it to say here that, unfortunately, nothing drives NDE R&D like a catastrophe. After this incident, significant R&D efforts were begun by the U.S. Department of Transportation, Federal Highway Administration (FHWA), to provide new NDE tools to bridge owners and inspectors in addition to the normal visual inspection done then and continued today. For example, as a direct result of the Silver Bridge disaster, work was begun at Southwest Research Institute (SWRI) on the Acoustic Crack Detector/Magnetic Crack Definer (ACD/MCD) system designed to inspect eye-bars and other fracture-critical structural steel members, i.e., those members whose failure could result in the loss of the entire bridge.

Recent statistics gathered by FHWA, and often quoted by others, indicate that over 40 percent of the nation's 578,000 highway bridges are "structurally deficient or functionally obsolete." About 30 percent of the more than 271,000 bridges in the federal-aid system, many of which are steel bridges, fall into this same category. Currently, FHWA's National Bridge Inspection Standards require that all bridge structures located on federal highways be inspected at least once every two years by a professional engineer who has been specially trained in maintenance inspection. Often these primarily visual inspections fail to detect cracks and other hidden defects which could seriously affect structural integrity. One significant problem is the limited access of many critical structural areas that often prevents or severely restricts visual inspections. Furthermore, the visual inspection problem is exacerbated by the need to protect fracture-critical, steel strength members against adverse environmental conditions by using protective systems such as galvanizing, paint, epoxy coating, external wrapping, and grout. Once structural elements have been fabricated and "protected," bridge owners are understandably reluctant to remove these protective systems or excavate grout for inspection purposes unless they know *where to look*. Even with protective systems, corrosion of steel members still occurs, and it is all but impossible to detect or locate corrosion under or within protective systems even with the recent advances in NDE technology.

The stated goals for FHWA NDE R&D are:

- Improved bridge safety for the traveling public
- Reduced "downtime" due to bridge loss
- Reduced maintenance costs through early detection of structural problems

Work toward these goals has been underway for some time, although at a relatively low level of funding. With the signing of the Intermodal Surface Transportation Efficiency Act of 1991 last December, new life has been brought to the FHWA's NDE R&D program by increasing the emphasis on NDE in the future development of inspection and monitoring systems and by fostering a close working relationship between FHWA and universities and industry to ensure bridge safety as well as public safety.

Under the new High Priority National Program Area for NDE, the objectives that have been set to achieve these goals are:

(1) Improve existing methods of NDE for quality control
(2) Improve local NDE used for identifying defects
(3) Develop reliable global NDE techniques

The *Conference on Nondestructive Evaluation for Bridges*, for which this paper was prepared, will help set the agenda and priorities for reaching these goals and objectives.

Thus, for over 20 years FHWA has actively sponsored NDE R&D projects to investigate new methods, techniques, and instrumentation systems that would provide state highway agencies and other bridge owners better tools to inspect and monitor highway bridge structures. Much of the work has focused on steel bridges. This paper attempts to put into perspective the state of the art in NDE of steel bridges in terms of both the methods practiced today (referred to as conventional nondestructive evaluation in this paper) and those more advanced methods and techniques undergoing R&D. Also, current R&D needs will be presented along with concluding remarks and observations suggesting future directions for the NDE R&D community.

2. BACKGROUND

Over the years, visual inspection has been the primary means of determining the condition of bridges and if more detailed procedures are needed. Conventional NDE methods such as liquid penetrant and magnetic particle testing have been applied only as an aid to the *detection* of cracks and other surface flaws during bridge fabrication, as well as during service. Radiographic testing has been the method of choice for detection of volumetric defects and is perhaps exceeded only by visual inspection in terms of frequency of use. Ultrasonic, acoustic emission, and eddy current testing* are relative newcomers but gaining in popularity. However, the fact of the matter is that these conventional NDE methods are currently used as an adjunct to visual inspection to provide additional confidence that cracks and other defects are detected which could affect bridge safety and performance.

Presently, there is no single NDE method that can reliably detect all the various types of cracks and defects known to occur in bridge steels (1),(2). Furthermore, in many situations, no single NDE method is adequate and a combination of methods should be used. Also, there are problems which no current NDE method can solve. Thus, the role of

*Although eddy current testing (ET) is emerging as an important NDE method for steel bridge inspection, because it does not require the removal of lead-based bridge paints and involve the associated risks, it is not widely used and not much information is available. Thus, it is not discussed further in this paper.

conventional NDE as practiced today is essentially one of supplementing or assisting thorough visual inspection, not one of substitution or replacement.

Successful NDE of steel bridges depends on three conditions being satisfied (1):

(1) The general area where defects are expected to occur must be determined *(where to look).*

(2) The optimum NDE method(s) for the area, type of defect, and specific inspection conditions must be specified *(how to look).*

(3) The method(s) must be performed by properly trained personnel *(who to look).*

Where to look is determined by those highly experienced in fabrication and in-service problems associated with specific bridge types. Typically, the "where to look" experts would have a structural engineering background or considerable bridge design and maintenance experience--normally not the NDE inspector. On the other hand, *how to look* requires in-depth knowledge of NDE methods and techniques--the purview of the NDE expert. Given where to look and the conditions of the inspection to be performed, the inspector must be capable of specifying the appropriate NDE methods or techniques. In fact, the lead NDE inspector should be an SNT-TC-1A Level III or equivalent. Lastly, properly trained personnel *(who to look)* are required to perform the specified method(s). Actual hands-on inspectors should be Level II or equivalent. [Although proper training is a crucial element of successful bridge inspection, the scope of this paper does not allow for in-depth review of training requirements. The reader is referred to reference (1) for more information on this subject.]

2.1 Steel Bridge Failures

There are three primary factors controlling the susceptibility of steel bridges to failure: stress, material toughness, and crack or flaw size (1). These factors may be interrelated by the well-known fracture mechanics expression:

$$K = C\sigma\sqrt{\pi a}$$

where K is the critical stress intensity factor, C is a constant that is a function of specimen and crack geometry, σ is the remote applied stress, and a is the crack or flaw size. Under static, plane-strain conditions, K becomes the fracture toughness of the material. Thus, fracture can be prevented by increasing the fracture toughness, decreasing the stress, or limiting the size of cracks and flaws. (Until relatively recently, however, fracture toughness was not taken into account, and many older bridges exist which were constructed with steels of unknown fracture toughness.)

For steel bridges, cracks are a major concern, and the detection of any cracks is a top priority before major damage or complete failure occurs.

2.2 Bridge Steel and Weld Defects

Cracks have occurred in steel bridge components largely due to poor quality materials, welds and weld/material-related problems, unanticipated stresses, and fatigue-prone connection details. Weld problems dominate and include, but are not limited to (1):

- Delayed cracking (hydrogen embrittlement, heat-affected zone, underbead, and cold cracking)
- Cracking at edges of butt-welded splices
- Lamellar tearing
- Improper backing-bar use
- Undersize welds--temporary welds/tack welds
- Weld repairs

- Weld discontinuities such as lack of fusion/penetration
- High hardness/low toughness

Fatigue cracks at welds, initiated by cyclic loading, are of particular concern. Prediction of fatigue strengths of bridge weldments is complicated by connection geometries, actual versus calculated stresses, and the presence of tensile residual stresses which can have a significant adverse effect on the fatigue strength of a given connection. Stress risers such as notches, gouges, weld undercuts, changes in cross section, and other discontinuities also can accelerate the development of fatigue cracks.

The American Association of State Highway Transportation Officials (AASHTO) has classified welded connection details according to their fatigue strength, and these categories are now used by bridge engineers to design new bridges and by bridge inspectors to set inspection priorities. Although the AASHTO classification is a significant step forward, it does not in any way alleviate the need for better NDE today and more advanced NDE in the future. Recent failures of bridges of modern design and construction, such as the I-95 Mianus Bridge in Connecticut (3), attest to the need for improved fabrication and repair techniques and for more effective quality assurance and quality control (QA/QC) inspection before, during, and after bridges are constructed and put into service.

Finally, the ability to detect and characterize cracks and other defects in bridge structures of all types (steel, concrete, wood) prior to the loss of structural integrity becomes increasingly urgent as bridges near the end of their expected service lives. This growing need defines a vital role for NDE R&D if this country is to stem the plague of the "aging infrastructure" and provide cost-effective maintenance, as well as fabrication inspection.

3. CONVENTIONAL NDE METHODS AND TECHNIQUES

Visual inspection of structural steel members and welds is the first step in any bridge inspection (1). Systematic written procedures are a must, with specific goals and objectives plainly stated, or the desired result may not be achieved. In addition, the visual inspection itself must be performed systematically by trained personnel using well established methodology. Some of the advantages of visual inspection are that it is relatively easy to perform, requires minimal equipment, usually does not take much time, and is more economical than more sophisticated NDE methods. Also, it detects obvious surface defects readily, and interpretation is generally straightforward. However, visual inspection has its limitations. Only surface defects can normally be detected; the size of the defect detected depends on individual visual acuity; lighting conditions, surface preparation, and viewing angle may be critical; and it is usually not appropriate for extremely small defects (less than 1/16 inch) or tight cracks. It is also important to recognize that visual inspection should be supplemented with liquid penetrant and magnetic particle testing when looking for fatigue cracks.

In addition to visual inspection, there are five conventional nondestructive evaluation methods routinely used today by bridge inspectors: (1) liquid penetrant, (2) magnetic particle, (3) radiographic, (4) ultrasonic, and (5) acoustic emission testing. These methods are described in detail in reference (1), and the reader is encouraged to consult this reference for more complete information than can be included here. However, each of these methods is briefly discussed in the following paragraphs with emphasis on steel bridge inspections, including their respective advantages and disadvantages. A methods comparison chart from reference (1) is presented at the end of this section (Figure 1). This chart may be used as a guide to selecting the best available conventional NDE methods. However, it is included in this paper primarily as an additional aid in defining the state of the art in conventional NDE of steel bridges. Most of the descriptive information and examples in the following paragraphs are from reference (4) on work done by the California Department of Transportation (CALTRANS).

3.1 Liquid Penetrant Testing (PT)

Aerosol (solvent-removable) penetrants in spray cans are widely used to determine the lengths and boundaries of cracks. For example, a majority of the fatigue cracks that have developed in California's steel bridges were initiated by out-of-plane bending (as is the case throughout the United States), and PT has proven to be a quick and reliable method for detecting these cracks.

Advantages of PT include:

- Simple to use and less costly than most other NDE methods
- Used on any nonporous material
- Very portable and well adapted for field use, particularly when using visible-dye, solvent-removable penetrants
- Very sensitive, particularly when using fluorescent penetrants

Disadvantages of PT include:

- Can only detect discontinuities which are open to the surface
- Requires surfaces to be properly cleaned to remove all surface contaminants such as paint, rust, scale, welding flux, etc.
- Requires good visual acuity of the operator
- Test sensitivity is reduced at low temperatures

3.2 Magnetic Particle Testing (MT)

Typically, dry magnetic particles are used with yoke magnetization to test fatigue-prone details. However, one drawback of this method is that it is not reliable in areas that have heavy paint buildup. In some cases, over one-half inch of paint has accumulated in areas to be inspected.

Advantages of MT include:

- Sensitive to small, shallow surface cracks
- Able to locate near-surface discontinuities with direct current magnetization
- Reasonably fast, cheap, and portable
- Little or no limitation due to size or shape of the part being inspected
- Ordinarily needs no elaborate precleaning

Disadvantages of MT include:

- Will work only on ferromagnetic material
- Will not disclose fine porosity
- Must have magnetic field in a direction perpendicular to the principal plane of the discontinuity for best detection
- Must take care to reduce overheating and arc strikes when using direct (prod) methods of magnetization
- The deeper the discontinuity lies below the test surfaces, the larger it must be to provide a readable indication and the more difficult it is to find
- For highest sensitivity, test surfaces should be cleaned and paint removed
- May need to remove residual magnetism
- Needs experienced and knowledgeable operators, a need often overlooked

3.3 Radiographic Testing (RT)

Conventional field radiographic testing with isotope sources is used, for example, to inspect welded connections on steel girder bridges and suspender rope sockets on suspension bridges. However, because of stringent safety requirements, this work is usually done by outside contractors who are specialists in the field. Very recently, portable, miniature, linear accelerators have been used to inspect large cable sockets.

Advantages of RT include:

- Permanent records (when film is used)
- Usable on most materials
- Able to detect internal flaws

Disadvantages of RT include:

- Is unable to accurately determine depth of the defect
- Is unable to locate defects perpendicular to beam
- May be difficult to radiograph complex geometries

- Is a safety hazard
- Requires accessibility to both sides of specimen
- Is relatively expensive, particularly with thick specimens

3.4 Ultrasonic Testing (UT)

Ultrasonics is used, for example, to inspect pin and hanger assemblies that suspend steel bridge sections over California's waterways and roadways. Pins are designed to rotate and resist shear and bending stresses, not torsional stresses. Over years of service, pins may become frozen due to corrosion; and the resulting torsional stresses can exceed the strength of the pin, resulting in failure. A longitudinal- or shear-wave ultrasonic test can detect pin cracking. Rivets and bolts can be similarly inspected. Also, ultrasonic-thickness gauging is performed on steel girders to determine section loss due to corrosion.

UT is most successful for detecting discontinuities oriented perpendicular to the direction of propagating sound. It is, therefore, an ideal complementary method when used with radiography. For optimum coverage, both methods, RT and UT, should be used.

Advantages of UT include:

- It has the ability to examine the internal structure of a material whose accessibility is limited to one side.

- Relatively thick specimens can be examined; bridge joint thicknesses rarely exceed 4 inches, and this is well within the range of UT.

- The method is readily adaptable to field testing; portable, lightweight units containing rechargeable battery packs are available for use with an 8-hour battery life.

- UT is an ideal method for detection of flaws which are generally not readily detectable by radiographic testing; laminar flaws, such as laminations, are commonly detected by the use of longitudinal-wave testing. Also, some forms of cracking are best suited for ultrasonics.

- Thickness measurements are easily performed using longitudinal-wave equipment; the equipment is now available in a digital display format and is very simple to operate.

Disadvantages of UT include:

- UT should not be used on rough surfaces, on parts with complicated geometries, on highly attenuative materials, or on materials where the discontinuity size is expected to be smaller than one-half of the wavelength.

- Other factors that limit the successful application of UT are lack of properly trained personnel, overestimation of the accuracy of flaw locating and sizing, and poorly written procedures.

3.5 Acoustic Emission Testing (AE)

Acoustic emission testing has not yet gained widespread routine use in bridge inspection. However, it is an emerging technology in its present form and is a fruitful area for further R&D.

Acoustic emission testing differs significantly from the other NDE methods discussed in this paper. Perhaps the most notable difference is that an applied stress is required to cause flaw growth and, hence, the acoustic emission. Unlike ultrasonic testing, the source of sound energy is the flaw itself. There is, therefore, no control of the AE intensity, and the AE transducers need only act as receivers. The applied stress can be the result of service and dead loads or an induced load used specifically for the AE test. In many tests, a combination of the two is necessary.

AE testing is a real-time NDE method. In other words, it is monitoring the actual condition of the component during the test. It cannot be repeated *exactly* because flaw growth is an irreversible process. When acoustic emissions due to a load which is causing crack extension eventually cease, further emissions will not occur unless the load is increased. This is attributed to the Kaiser effect.

The AE test method can be used to record an accumulation of damage occurring within a structure. The data obtained can be used as a history of a structure and possibly to predict failure. Other NDE methods measure the degree of damage incurred at some previous time; AE detects that which is potentially most serious, i.e., defects which are still growing. Cracks on the order or a millionth of an inch can be detected if they are actively growing. Conversely, a huge crack can go undetected if it has stopped propagating. The AE test method is also being used during welding to detect weld discontinuities as they are being made.

Detecting the release of sound energy with AE test equipment is a fairly straightforward procedure. Many successful experiments under both laboratory and field conditions have been conducted.* However, a primary limitation of AE testing of structures such as bridges is in discriminating the sound energy released by a growing flaw from that which is called background noise. Many background noise generators--bolts, joint friction, traffic, and others--can mimic or mask the sound energy released from growing cracks. Some AE test methods avoid this problem by isolating areas known to contain possible background noise generators.

*Laboratory work conducted in the early 1970s showed that the fracturing of individual wires in steel cables could be readily detected using AE. Results showed that continuous AE monitoring provided an early warning before a fatigue failure by as much as half the overall fatigue lifetime. *Proof-testing* of wire rope using AE also provided warning signals of approaching fatigue failure. Cables of the Dumbarton lift bridge near San Francisco were subjected to AE tests. These cables were tested for 10 minutes by providing a transverse load with a hand winch. Although several of the cables showed more emission activity than others, none of them was deemed to be seriously deteriorated. However, attenuation in the cables was high.

In summary, as stated in an FHWA-sponsored study of AE test equipment for bridges, "If AE technology is to make the transition from the laboratory to practical application on bridge structures, a method is required that is capable of *reliably classifying* emission *source, type,* and *severity.*" Although many of the early limitations have been overcome with the advent of computerized signal analyzers, careful sensor placement and system calibration are necessary. With the emergence of more refined equipment and software packages, stand-alone systems requiring little user input are being developed for specific purposes.

Advantages of AE include:

- Its ability to monitor responses of complete structures to applied loads.

- By observing acoustic emission behavior during proof-testing or on in-service structures, it is possible to identify and characterize material degradation as a function of load in real time.

- AE can be performed with little or no downtime.

- The detection of *incipient cracks* makes AE an excellent complement to other NDE methods such as radiography and ultrasonics, since it can tell if the crack is growing or not.

- AE is a continuous monitoring technique; by using stationary unmanned units, on-line monitoring of structures can be performed using electronic data storage equipment for long-term evaluations.

- AE can evaluate components that may be inaccessible to other NDE methods or techniques.

Disadvantages of AE include:

- AE may not be able to separate relevant crack-related AE signals from non-relevant background noise; users of AE technology must be aware of outside acoustic emission sources, which can be detected by the transducers.

- Computerized filtering schemes are electronic intensive and require a high level of expertise to conduct and plan.

- The defect must generate its own signal; in other words, a means of loading a structure is essential since acoustic emissions are a result of stress-created molecular displacement. Underlying this limitation is an inherent *advantage* in that it is only the growing defects that are revealed. It should be pointed out that for bridge applications, the normal peak traffic loads are usually sufficient in providing the load stimulus needed.

3.6 Comparison and Selection Criteria

The "NDT Method Comparison Chart for Steel Bridges" shown in Figure 1 can be used as a guide in the selection of a conventional NDE method for the detection of the various discontinuities occurring in steel bridges (1). It must be considered only a guide, however, since the specific circumstances of the test (accessibility, surface condition, temperature, etc.) can significantly affect the results of any NDE method. It is also very important to use the notes in conjunction with the chart.

In conclusion, visual inspection has been and continues to be the most popular bridge inspection method, and there is nothing wrong with it when properly applied. It may be the most effective and economical approach overall. However, there are many situations when visual inspection is just not enough due to its obvious limitations, and it must be supplemented with at least one of the more sensitive conventional NDE methods (if one exists) or with a more advanced NDE method when required. The challenge to the NDE R&D community is to provide the advanced methods and techniques.

4. ADVANCED NDE METHODS AND TECHNIQUES

Most of the recent advances in nondestructive evaluation applicable to steel bridges have been funded by FHWA. However, several have been funded by various private organizations and companies under internal research and development programs. In general, all the developments may be classified or grouped into one of four categories: (1) local flaw detection; (2) global monitoring; (3) residual stress measurement; and (4) signal processing, analysis, and display methods. Within each category, several developments have occurred, and *representative* examples of these are discussed in this section mostly by technology area, e.g., ultrasonics, electromagnetics, etc. Within a given category, the representative developments are discussed in approximately chronological order.

4.1 Local Flaw Detection

4.1.1 Ultrasonics

Acoustic Crack Detector. One of the earliest developments funded by FHWA was the Acoustic Crack Detector/Magnetic Crack Definer (ACD/MCD) system shown in Figure 2 (5). Work on this system was begun by SWRI shortly after the collapse of the Silver Bridge. The discussion in this section is limited to the ACD subsystem although the ACD and the MCD were combined into one system development for contract purposes. The MCD is discussed later.

The ACD is an ultrasonic system essentially consisting of an electronic backpack and a hand-held ultrasonic probe; the backpack provides excitation to and also electronically processes signals from the probe. The probe can be operated in either one of two modes by appropriate positioning of a switch on the probe.

In the S position, or "Survey Mode," two separate ultrasonic transducer channels operate alternately. One is a 5-MHz transducer that transmits bursts of ultrasonic energy in a direction perpendicular (compressional wave) to the inspection surface. This energy travels through the steel, is reflected by the back surface of the bridge element, and returns to the transducer. As long as the reflected energy received by the transducer is above a threshold level, a green light is illuminated on the probe, indicating that sufficient energy has been coupled into the steel to permit reliable inspection. The other is a 2.25-MHz transducer that is inclined at an angle to the surface and transmits bursts of ultrasonic energy (shear wave) in a direction such that the wave is reflected repeatedly from one surface to another as it travels progressively down the length of the bridge member. When the ultrasonic energy encounters a target such as a fatigue crack, bolt hole, or end of a beam, the energy is reflected back along the path and into the transducer. The round-trip travel time is electronically measured. This time is translated into distance and displayed by a digital readout on the probe indicating to the operator the distance from the probe to the target in feet and tenths of feet. As the probe is maneuvered to survey a bridge member, the distance to fatigue cracks and other flaws is continuously displayed on the digital readout; the digital readout decreases with decreasing distance to the target and increases with increasing distance to the target.

When the switch on the probe is placed in the "L" or "Lamination Position," only the 5-MHz transducer that transmits bursts of ultrasonic energy in a direction perpendicular to the inspection surface is utilized. The energy enters the steel and, on encountering either the back surface of the bridge element or an internal lamination, the energy is reflected back into the transducer. The round-trip travel time is measured and electronically translated into distance. This distance is displayed on the digital readout in inches and tenths of inches. As long as this distance coincides with the thickness of the bridge member, no laminations exist; however, when the digital reading is less than the total thickness, a lamination has been encountered and can be mapped by appropriate maneuvering of the probe.

The ACD survey probe can be used to inspect eye-bars for cracks radiating from the bore provided the sides of the eye-bar are accessible. However, for those situations where access to the side surfaces is seriously restricted, special ACD eye-bar probes are available to permit inspection of the bore region from the periphery. This eye-bar probe uses dual transducers, as shown schematically in Figure 3. Transducer A transmits pulses normal to the periphery of the bore, and the reflected signals provide a coupling signal. Transducer B injects ultrasonic energy at such an angle that the beam grazes the bore hole tangentially, intercepting cracks radiating from the bore hole. A rather broad ultrasonic beam can be used in Channel B so that angular alignment of the probe on the periphery is not highly critical. A broad beam also better accommodates the geometry of typical eye-bars.

To inspect regions with limited access, the ACD eye-bar probe has a total width of 0.9 inch so that it can be inserted into 1-inch-wide gaps. The eye-bar probe consists of an inspection head mounted on a tube approximately 10 inches long attached to the upper sections of a conventional ACD survey. The complete downhole subsystem (Figure 7), including carrying cases, weighs only 25 pounds and allows the operator freedom to climb using both hands. The hand scanner is a linear optically encoded X-Y scanner with an aperture size of 11 × 7.5 inches and an imaging resolution of 0.90 inch. The transducer is mounted in the scanner, and the operator maneuvers the transducer about the area of interest. The operator maintains good transducer contact and is in constant control of the equipment. Numerous types of transducers and configurations for straight- and angle-beam UT techniques can be accommodated by the scanner and uphole subsystem. The video monitor is a DC-powered, black-and-white, 5-inch television that can be switched by the downhole operator to display an A-scan signal or a computer-generated image, permitting the operator to see which scanner position produces the best results. The pre-amp boosts the reflected signal response to a level where it can be transmitted through the 500 feet of cable without interference. Both subsystem operators coordinate their actions by using the audio line for verbal communications. The downhole headset contains the microphone and earphones in a single "hands-free" unit.

Once the UT signal leaves downhole, it enters the receiver module uphole where it is displayed as an A-scan on the oscilloscope, then relayed to the gate module where the signal is gated (Figure 9), and finally proceeds to the signal processor. The signal-processing module is the most important element of the system. It takes signals generated by the interaction of ultrasonic waves with the interior of solid objects and uses computer processing to convert these signals into color images that are displayed on a monitor.

Hands-Free Ultrasonic Test View. A very recent development by Sierra Matrix, Inc., that appears promising and may prove very useful for steel bridge inspections is the Hands-Free Ultrasonic Test View (H-F UTV). According to the product description on this device, the H-F UTV is a portable, battery-operated, real-time, ultrasonic RF signal recorder (see Figure 10) (6). An innovative user interface permits the entire instrument to be worn, instead of carried, and operated without hands. This capability is possible because the system can be trained to recognize simple voice commands and to display all test feedback on a small, 4-ounce, virtual-image, heads-up display. Therefore, the H-F UTV can be used for inspection situations that present logistical problems of access or operation such as occur when an inspection requires one or both hands for climbing and support or for inspection within confined spaces having insufficient room to operate the controls or view the display of a standard UT instrument.

The H-F UTV recognizes (speaker-dependent) natural speech commands in any language and will immediately execute commands from any menu. For example, the command "plus six dB" increases the receiver gain by 6 dB, and "point 1 inch delay" sets the start of a gate to 0.100 inch. The high-contrast LED monocular display can be seen simultaneously with the part surface. The system supports timed and on-demand data acquisition modes. The state of all controls, or "setup," is fully programmable and is recorded with each data set. A window-based user interface also supports a serial pointing device and color ink jet printer.

Benefits include:

- Portable, battery-powered RF data acquisition with square wave pulser
- Repeatable, fully documented inspections using standard recording format
- One-person operation

Applications include:

- Bridge inspection: eye-bars, box beams, backup bars, etc.
- Weld inspection
- Corrosion, erosion, and thickness mapping
- Flaw detection and sizing
- Data capture for off-line postprocessing

Although this new device has not yet received widespread use for steel bridge inspection, it embodies many of the original concepts envisioned in the FHWA's ACD system and in the later UBIS, while getting around some of their limitations; and it should be considered for evaluation by state highway agencies.

4.1.2 Electromagnetics

Magnetic Crack Definer. As mentioned previously, the Magnetic Crack Definer (MCD) was originally developed by SWRI as a subsystem of the FHWA's ACD/MCD system. The MCD also consists of an electronic backpack and a hand-held probe (see Figure 2). The purpose of this subsystem is to precisely locate and define the length and ends of cracks, usually located by the Acoustic Crack Detector (ACD). In some cases, it may be used to survey limited regions to detect cracks. In operation the probe magnetically excites a small region of the bridge member, and signals resulting from magnetic gradients are continuously analyzed electronically. As the probe is slowly maneuvered to survey the suspect region, characteristic signal changes occur as fatigue cracks or the edge of steel plates are encountered. The crack location and direction are then indicated by automatically illuminating one of two elongated red lights; the long axis of the light indicates the direction of the crack. As long as the probe is in the immediate vicinity of the crack, the light will remain illuminated; however, if the probe is moved off the crack, the light is turned off. One detection channel, operating at a frequency of 105 Hz, is designed primarily for locating cracks in the toe of a weld and large open cracks; the other channel, operating at a frequency of 315 Hz, is primarily designed for locating tight cracks and determining the location of a crack tip. Subsequent additions and modifications to the original MCD have included: (a) smaller probe size, (b) proximity tone, and (c) improved power supply (AC operations with charging indicator for batteries).

Magnetic Field Disturbance System. Strictly speaking, the Magnetic Field Disturbance (MFD) system was developed by SWRI to inspect steel-reinforced concrete bridges, but it is discussed here because it was the forerunner to the FHWA's prototype Magnetic Perturbation Cable (MPC) inspection system developed by Texas Research Institute, Inc., (TRI) primarily for steel wire and strand, cable-stayed suspension bridges, that is undergoing further development by TRI at this time as the Magnetic Flux Leakage Inspection System (MFLIS). Both of these newer developments are discussed later.

The MFD system was designed to detect anomalies such as prestressing strand and rebar fractures in the steel members of reinforced concrete bridge beams using the magnetic perturbation method (7),(8). The system comprises an electromagnet to produce a magnetic field locally in the volume to be examined; the interaction of the magnetic field with the steel reinforcing elements is sensed by an array of Hall-effect probes. The electromagnet and the probe array are transported as a unit along the steel-reinforced beam to scan regions of interest. Anomalies or discontinuities in the steel elements, such as local corrosion and strand and bar fractures, cause a perturbation of the magnetic flux which is sensed by the probe array and recorded.

Initial investigations were conducted using arrays of Hall-effect elements scanned beneath the lower surface of a test beam. Based on the results from both transverse and vertical sensor arrays, it was originally determined that the most promising configuration was a four-element array in which three of the elements were spaced transverse to the beam and the fourth was located below the center element of the three. To investigate the combined use of the four-element array and signal analysis techniques, modifications were made to accommodate various signal processing algorithms.

Recently, Ghorbanpoor et al. independently evaluated the MFD system (9). This evaluation was divided into four primary tasks: (1) laboratory tests of bar and strand specimens that contained mechanical flaws, (2) accelerated corrosion studies, (3) field tests, and (4) limited investigations of methods for reducing the size and weight of the original MFD instrumentation. Four types of bar flaws were investigated including complete fracture. Strand flaws consisting of fracture of one or more wires with varying gap widths were also studied. Flaw signal characteristics were studied for a variety of configurational conditions and were found to be dependent primarily upon the size and depth of the flaw and, importantly, the proximity to stirrups. Based upon this and previous studies, as well as the literature, the magnetic flux leakage method employed in the MFD system was concluded to be currently the most promising method for identifying flaws in reinforcing and prestressing steels in concrete. Flaws as small as 7-percent loss of section were detected when they were located in the closest layer of reinforcement to the bottom of the girder and outside the influence of stirrups.

Three basic limitations were identified with the original MFD system: (1) the correlation method of signal analysis has limited flaw discrimination capabilities primarily due to the masking effect of stirrups cast into the girders; (2) the electronic hardware is outdated, resulting in significant amounts of time required for both data acquisition and analysis; and (3) the excessive size and weight of the MFD system make routine inspections very impractical. Replacement of the electromagnet with a lighter permanent magnet was recommended. The resulting inspection cart and supporting hardware would be significantly lighter, improving handling efficiency in the field.

In addition to the findings above, a number of other important limitations were encountered with the present MFD system. For example, it was only effective for detecting flaws with greater than approximately 33-percent loss of section when located midway between the stirrups of a test strand and in the lowest layer of reinforcement. When flaws were positioned coincidentally with stirrups, only flaws of 53-percent reduction or larger were detected in the lowest layer. In a number of cases, very large flaws, such as complete fractures, were not detected at all when located deeper than the first row and in a region containing rectangular stirrups. In some cases, very strong flaw indications were obtained in locations where flaws were known not to exist. These false correlations usually occurred in the regions between stirrups. Although the subtraction or "differencing" technique developed by SWRI for the MFD system was a significant improvement to the data analysis, it could also be the cause of the false flaw indications obtained. Conceptually, there are two problems with the differencing method: (1) the subtraction inevitably reduces the flaw signal amplitude along with the stirrup signals and (2) since one of the four field sensors is located at a greater vertical distance from the flaws than the lower three, the signals recorded by this sensor will have greater peak-to-peak separations. The net effect of the differencing will be to introduce extraneous signals into the resulting data, particularly if the data have been normalized in amplitude prior to the subtraction. This may generate false flaw indications when the correlation method of signal analysis is used.

A number of factors affect the properties of the flaw signals and to reduce flaw discrimination capabilities of the MFD system. The most important of these include:

- Flaw size
- Flaw depth
- Presence of stirrups

The presence of additional longitudinal steel was found to significantly reduce flaw signal amplitudes, especially when the added steel was located between the sensors and flaws in deeper bars.

A new signal analysis approach was evaluated to overcome the limitations of the correlation method and to minimize the adverse influence of stirrups in flaw signals. This approach, the profile method of analysis, is more dependent upon signal amplitude and spatial distribution and less dependent upon shape. Initial work indicated that the Magnetic Field Profile (MFP) technique is promising as a complement to the correlation method. In addition, a comparative study of MFD records over time (i.e., conditioning monitoring) was examined during the corrosion study and was found to significantly improve results of the correlation method of analysis. This approach to periodic MFD evaluation of a structural member and comparison to prior records, combined with the MFP and correlation analysis methods, was found to be the most reliable approach to flaw detection.

Magnetic Perturbation Cable Inspection System. FHWA sponsored research at TRI in the mid-1980s to investigate the development of a prototype NDE system for inspecting and monitoring structural cables and strands of suspension bridges (10). The magnetic perturbation or flux leakage method was selected for this cable inspection problem. Although this method had been previously applied successfully by others to a variety of NDE problems, the application to much larger diameter cables was an extremely difficult endeavor.

A limited parametric investigation was conducted using a rudimentary system consisting of a 200-pound electromagnet with an attached Hall-effect probe to measure the magnetic perturbations as a simulated cable was scanned. The first specimen consisted of 370 steel rods, each 0.25 inch diameter by 20 feet long, enclosed by a 6-inch ID by 0.28-inch wall steel pipe 20 feet long. The steel pipe supported the rods and also simulated the 0.196-inch-diameter steel wire spiral wrap used on some main cables. However, magnetic anomalies in the steel pipe obscured flaw signals except for very large flaws; specifically, a 0.8-percent cross-sectional area flaw could be detected to a depth of 1 inch, but a 1.5-percent flaw could not be detected at a depth of 2 inches. The steel pipe was replaced by an aluminum pipe, and eventually it was possible to detect a 1.08-percent cross-sectional area flaw [signal-to-noise (SNR) ratio = 10:1] at depths beyond 1.7 inches, but not at 2 inches. An improved sensor, changes in filtering, and increased magnetization improved flaw detection significantly; detection of a 0.27-percent cross-sectional area flaw at a depth of 1.9 inches was achieved. Subsequently, a prototype system was constructed, and flaw detection capability has been confirmed on a 20-foot-long specimen simulating a segment of the 6.3-inch-diameter Luling Bridge stay-cables. Specifically, detection of a 1.5-percent cross-sectional area flaw has been verified at a depth of approximately 2 inches (SNR = 30:1). It is estimated that one fractured wire (0.37 percent) of the 271 wires in the Luling Bridge stay-cables could be reliably detected at a depth of 2 inches with an SNR of 4:1.

Field demonstrations of the prototype MPC system were conducted on the Luling Bridge stay-cables during the period December 12-15, 1988. Demonstrations were successful, but a few malfunctions occurred. Approximately 70 scan sequences or 420 data tracks (each 100 inches long) were recorded. Comprehensive analyses of many data sets did not disclose any signals indicative of wire fracture. However, signals and "features" on many data sets indicated: nonuniform thickness of the cement grouting; the possibility that, in several regions on top of the cable, grouting was so thin that segments of wires were moving in response to the attractive forces exerted at each pole face; and that pitch of the 1/4-inch-diameter rope spiral wrap was only 25 to 35 inches rather than 6 feet, as specified on engineering drawings from the stay-cable manufacturer. Limited additional experiments on a modified laboratory specimen, with decreased magnet-to-steel bundle distance that approximately doubled the flux density in the steel, increased flaw signals by approximately 10X with no increase of noise. A 0.37-percent cross-sectional area flaw (0.25-inch gap in one 0.25-inch-diameter wire of the 271-wire bundle) located in the second layer (or 0.75 inch from the electromagnet) produced a signal amplitude of approximately 1000 mV, and noise was less than 20 mV.

Magnetic Flux Leakage Inspection System. The Magnetic Flux Leakage System (MFLIS) (Figure 11) is an outgrowth of the earlier prototype MPC system that was built only for cable inspection and the older MPC system for inspecting reinforced concrete beams. In fact, the MFLIS combines the functions of both into one system that is designed to inspect cables and reinforced concrete beams alike. The newer MFLIS is currently being built by TRI under FHWA funding and is expected to be ready in about a year for field trials.

The MFLIS employs a modular design with interchangeable permanent magnet/Hall-effect sensor modules, shown in Figure 12 (11). These modules are mounted on a variety of carrier/transport mechanisms to inspect both cables and beams. For main suspension cables, the transport mechanism is designed to negotiate suspender rope cable bands without having to be lifted, for example, with an auxiliary crane from one cable panel to the next. (For stay-cables, there are generally no obstructions such as the cable bands.) In concept, the MFLIS will inspect the entire length of a side-span or mid-span in one pass since the magnet/sensor modules are arrayed radially (adjustable to various diameters) to interrogate the full circumference (see Figure 13). For reinforced concrete beams, the same single-pass inspection is envisioned with the modules in a flat array (adjustable to various widths) under the lower flange of the beam (see Figure 14).

The size, weight, power, etc. have been reduced significantly vis-a-vis the MPC system by going to a modular permanent magnet/sensor design; and the overall time for an inspection should be minimized with single-pass inspection. However, since magnetic flux leakage is, by nature, a localized phenomenon, the depth of cable inspection is limited (on the order of a few inches); and the inspection under cables bands, over tower saddles, around turning shoes, and in anchorages is not practical. Unfortunately, much of the corrosion damage and wire fractures observed in cables is in these areas. Also, bridge traffic will have to be curtailed and controlled during an MFLIS inspection, and one lane will have to be closed to accommodate the operations van, trailers, winches, etc. on the bridge deck (Figure 15). Notwithstanding these shortcomings, however, the MFLIS does represent a significant step forward in the automated scanning inspection of steel bridge cables and reinforced concrete bridge beams.

4.1.3 Computed Tomography/Real-Time Radiography

Photon Tomography. In bridge fabrication, the NDE acceptance of weldments has been based on UT and RT inspections and evaluations. The inspection of weld details is in accordance with the American Welding Society D1.1. Structural Welding Code. It is well recognized that both NDE methods have certain limitations and may not detect a flaw or, if detected, may not indicate its severity. To address this problem, an FHWA research study was conducted by Scientific Measurement Systems, Inc., (SMS) to assess the potential use of photon tomography for weld inspection in the field on typical bridge members (2),(12). Instrumentation was developed and assembled for this investigation.

A tomograph is a determination of the cross section of a solid from a series of radial scans around the solid. An isotope gamma-ray or an x-ray source used in conjunction with solid-state detectors and signal processing hardware make up a photon tomography unit. Detected signals are processed digitally and reconstructed as a cross section of the member on a cathode-ray tube. The basic information conveyed is the density of the material in the section scanned. Weld defects represent a sharp density contrast and are easily detected. Digital processing allows not only for the construction of the tomogram but also signal enhancement by correcting for scattering and presenting iso-density contours of defects and interfaces. The result is a two-dimensional density representation of the object that, along with other scans, can be used to construct a complete three-dimensional density image. Ultimately, however, the application of tomography to field inspection requires that a method of developing a tomograph using a partial scan--not a full rotation about the object--be studied. As a result of this project, a prototype tomographic scanner design was developed for equipment that could be used immediately in the fabrication shop.

Examination of Steel and Welds. In the final report, "Application of Photon Tomography to Inspection of Bridge Weldments" (15), it was determined that computed tomography (CT) scanning can be a cost-effective tool in examining welded components during production. Although the images of cracks in these tests were not of the quality expected, CT scanning was determined to be a valuable quality control technique when used in conjunction with other nondestructive methodologies such as UT and conventional RT. CT was able to image cracks that were not detected using the most sensitive RT equipment. It also determined when UT was providing false indications. Figure 16 shows a tomogram taken of a defective butt weld. Notice the dark area at the center of the weld indicating a void in the weld at this scan point.

Because steel is very dense, tomographic examination of welds requires high-energy beam sources such as Ir-192 or Co-60. About one-half of the beam emanating from an Ir-192 source is scattered or absorbed by each 0.4 inch of steel. This limits its penetrating power for CT use to approximately 5 inches of steel. Co-60 can be used on steel up to 10 inches thick. Examining steel in greater thickness requires the use of very powerful and very expensive x-ray generators. For example, a 16-MeV source CT scanner was developed for the Air Force and is one of the most powerful CT systems in the world. It can accurately scan missiles up to 8 feet in diameter, 17 feet long, and weighing up to 55 tons. This machine demonstrates the present outer limits of CT use on steel components.

Examination of Cables. A laboratory CT scanner was recently employed by SMS to examine cables and sockets commonly used on many suspension bridges in the United States (13). Figure 17 is a drawing of one of the sockets examined during this test. Note the improperly splayed strands and foam-formed voids intentionally placed in this test specimen as a control measure. The cables ranged in diameter from 1.375 to 1.625 inches, and the sockets were 5 to 5.5 inches in diameter. The specimens used in these experiments were prepared by the Kentucky Transportation Research Program (KTRP) at the University of Kentucky and utilized a prototype machine developed by SMS. Data gained from these tests have been used in a preliminary CT system design suitable for the on-site inspection of bridge suspension cables and fittings.

Simulated imperfections within the specimens were calibrated in order to measure the spatial resolution and quantitative accuracy of CT scanning. Various diameter holes drilled in one specimen were correctly sized down to 0.04 inch, demonstrating the resolution capability of CT scans in measuring voids within dense materials. The tests also revealed a high level of porosity in one fitting removed from a bridge crossing the Ohio River. Voids were clearly visible, indicating that the fitting had been cast, not forged, as was originally presumed. Other flaws, intentional or not, were precisely located and sized with better-than-expected results. These tests, along with others, have substantiated the potential use of CT technology for nondestructive on-site inspections of structural steel and suspension cable bridges.

Advantages and Disadvantages. Computed tomography is unsurpassed for accurately imaging, measuring, and inspecting the interior of objects and has many advantages over conventional radiography. The CT image yields a true cross section of an object, free from the superimposed features above or below the area of interest. Visual noise is, therefore, essentially absent in CT images when compared to radiographs.

Another important advantage of CT is the resolution of low-contrast images over a large area. Standard x-ray films are normally capable of distinguishing a 1- to 2-percent variation in material density. CT density resolution is on the order of 0.1 to 0.5 percent, a marked improvement over x-ray films. CT can routinely resolve details down to 0.050 inch, and small cracks and imperfections down to 0.002 inch can be discerned under ideal conditions.

Unlike standard radiography, which offers only qualitative data, CT offers both qualitative and quantitative information. Quantitative CT data allow for automated quality control measurements to be made. Since all CT images are constructed from digitally encoded information, manipulating the data by computer is easily accomplished. By filtering out unwanted signals, enhancement procedures can be employed to clarify internal density changes not detected using other NDE techniques. Another advantage of having information in a digitized form is that inspections can be performed automatically. Image information from an object is compared to a standard image of how the object should appear. Defective parts can be quickly spotted before they are used in the final assembly. Thus, near 100-percent quality-assured components are possible.

The main drawbacks of using CT technology for bridge inspection purposes are the initial expense and lack of portability. Current CT systems start at $500,000, are heavy, and are designed primarily for stationary use (except for the IRIS™ discussed next). Although the initial cost factor is high, it can be soon outweighed by the savings these systems offer. In one instance, a tire company estimated that a savings of $700,000 could be achieved annually using CT scans as opposed to destructive testing, which involved the cutting up of numerous perfectly good tires. New prototype systems are aiming towards portability by reducing the size of the components and by developing software to eliminate background noise. All in all, CT technology appears to have a bright future in NDE inspections of highway structures.

Integrated Real-Time Inspection System. International Digital Modeling Corporation (IDM Corp.) has developed a system called the Integrated Real-Time Inspection System (IRIS™) incorporating CT and real-time radiography (RTR). Based on information from the system brochure (14), IRIS™ has the ability to detect critical dimensions and defects in metal products in real time. IRIS™ also provides quantitative dimensioning and imaging capabilities not possible with other gauging techniques. The IRIS™ technology provides the capability of performing dynamic digital measurement and inspection of tubular products, *in situ*, such as on-line piping in the electric power industry, processing lines in the chemical industry, and tubular products in steel mills during their manufacture. Importantly, IRIS™ has potential application to steel bridge cable inspection and monitoring.

IRIS™ is engineered to provide full capability for dimensioning and imaging and is currently in use to integrate digital real-time data analysis for industrial process control and monitoring.

- *3-D Analysis of On-Line Piping.* The volumetric analysis of pipe under operating conditions, insulated, and at elevated temperature and pressure, using IRIS™ technology, is unique in the process monitoring/industrial inspection field.

- *Minimal Consideration of Object Geometry.* Profiling and sizing objects requires a volumetric coordinate measuring and quantifying ability. These features are inherent to IRIS™ such that surface features such as pitting, corrosion, and wear are identified and imaged.

- *Complementary Detection/Measurement Capabilities.* Conventional NDE methods such as RT and UT are classified with respect to defect detection as volumetric-sensitive and planar surface-sensitive, respectively. IRIS™ is capable of detecting both volumetric and planar surface flaws.

- *Archival Data and Post-Analysis Techniques.* A valuable feature of IRIS™ is its ability to track pipe parameters such as creep, crack growth, wall thinning, and corrosion over time and thus help assist utility practices and preventive maintenance. This attribute is unique in the NDE field.

- *100-Percent Inspection Option.* A plant outage is not required for IRIS™ to perform full-length pipe inspection, including girth welds. This option is not feasible with other current techniques, hardware, and practices.

4.2 Global Monitoring

4.2.1 Acoustic Emission

Acoustic Emission Studies. Early acoustic emission tests sponsored by the FHWA on in-service bridges yielded few useful results (2). The main problem was to distinguish acoustic emission events emanating from defects from signals due to background noise. Typically, noise is from traffic, rain, and *fretting*. Fretting noises are those produced by the rubbing action, originating mainly from joints fastened with rivets or bolts. These noises, along with the use of only a single transducer, rendered many of these early tests inconclusive. Later research showed that one method of overcoming these problems is to use guard transducers to reject noises coming from sources outside the area of primary concern. In this setup, background signals are rejected if they strike one of the guard transducers before they reach one of the test transducers.

Acoustic Emission Weld Monitor. While much of the time devoted to bridge inspection is spent in the laboratory developing and testing techniques and innovations, it still remains that the equipment developed must be applicable to field inspection--highway bridges simply are not erected in laboratories. However, the fabrication of some bridge components does take place in various plants; and it is recognized that the detection of flaws, especially in weldments, at the fabrication plant offers considerable advantages in costs and time, to say nothing of the risks involved in erecting structurally deficient members. To address this need, an acoustic emission study was initiated by FHWA with Gard, Inc.; and an instrument was developed to detect, locate, and monitor flaws during the welding process (2),(15).

The Acoustic Emission Weld Monitor (AEWM) built by Gard (Figure 18) uses three microprocessors and three analog input channels. Dual floppy disk drives and a computer video terminal are used to augment the operation. This allows the recording of raw AE data on disk for post-weld analysis and, in addition, permits the operator to modify the processing parameters to allow optimization of monitor performance for a particular set of weld conditions. The AE signals are subjected to three successive tests--energy or ringdown count limits to reject AE bursts which are either too low or too high in energy. The rate and location criteria are based on the fact that a flaw generally produces higher rate AE than other sources, and the flaw-related events will all come from one localized point rather than be scattered.

The AEWM may provide immediate aid to fabricators in several important areas: (1) improving repair decisions, (2) detecting flaws, (3) prequalifying weld procedures or welders, (4) reducing material handling in the shop and welder downtime due to conventional NDT inspections, and (5) reducing or eliminating post-weld repairs.

In the early 1980s, FHWA contracted with Gard, Inc., to test the AEWM on typical submerged-arc, butt-welding operations (2),(16). Laboratory tests were performed on grooved plates of typical highway bridge steels. In their laboratory tests, typical weld flaws such as cracks, porosity, slag inclusion, and lack of fusion were deliberately created in the welds as they were being formed. A total of 24 planar discontinuities (cracks and lack of fusion), six slag inclusions, and eight porosity defects were generated. The AEWM successfully detected all of the planar and slag inclusion discontinuities, and seven of the eight porosities were detected. Following this laboratory success, the system was field-tested at three fabrication shops. In one of these trials, a total of 690 feet of welds was monitored. The system detected a total of six minor discontinuities that were confirmed. No other major discontinuities were detected by the AEWM or by radiography.

Recently, the Kentucky Transportation Research Program (KTRP) completed a 10-month-long evaluation at the High Steel Structures, Inc., fabrication shop in Lancaster, Pennsylvania (2),(17). During this study, the AEWM was used routinely to test web and flange butt-welding operations. A total of 153 welds were monitored which consisted of 736.2 feet of completed welds. Prior to welding, the AEWM transducers were attached and the AEWM calibrated. Each weld pass was monitored as it was being deposited. If the AEWM indicated discontinuities, the weld was visually inspected for flaws. Results of each weld pass were forwarded for future correlation with radiography results by shop quality control personnel. Thirteen percent of the AE indications were "valid" indications. Three of the "valid" indications were confirmed by conventional NDE. Only one of those, a slag inclusion 3 inches long, was classified as code-rejectable. Effective steps have been instituted to eliminate the large number of overcalls and yet still retain the unit's ability to accurately detect code-rejectable defects.

Examination of Structural Steel. As mentioned above, Gard originally developed the AEWM (Figure 18) for in-process weld monitoring. However, since 1982, KTRP has been using this system to detect crack activity on steel bridges in high mechanical noise environments (2),(18). The unique feature of the AEWM is the microprocessor-based filtering system. This system is able to analyze multiple AE parameters, reject noise-related activity, and locate and characterize defects in *real time*. KTRP and Gard personnel have used the AEWM on nine bridge sites in four states. Bridge details such as floor beams, horizontal and vertical stiffeners, pierced box girders, stringer attachments to deck beams, upper and lower web, flange welds, double cantilever box beams, tie chords, etc. have been monitored.

On the I-24 bridge over the Tennessee River, the AEWM was used to monitor cracks in the webs of floor beams at connections with the tie chords. Two transducers were used to locate cracks, and since the sites were near bolted splices, high background noise was experienced. Tests were conducted during normal traffic that was the load stimulus needed to produce acoustic emissions from crack growth. Although five crack sites were examined, each for a period of two hours, only one of them provided high emission activity. The AEWM was able to characterize emission activity located at the crack tip. In order to observe the growth of cracks, three more tests were conducted at the site where high activity was observed. These secondary tests indicated a decrease in the acoustic emission activity with time. This was explained by the fact that out-of-plane cracks tend to be self-extinguishing.

Experience on eight other bridges showed that the AEWM was able to reject high background noise levels typically encountered when inspecting in-service bridges. Further, these tests demonstrated that the AEWM can detect fatigue-crack activity in steel bridge members.

Acoustic Emission Monitor. One of the most recent developments, coming from FHWA-sponsored research is the Acoustic Emission Monitor (AEM) (19). This rugged micro-computer-based system uses firmware signal processing algorithms which allow flaws to be detected, located, and characterized in real time in high background noise environments. These algorithms allow a detected flaw to be identified as to type in one of five categories and provide a relative indication of flaw severity. Up to six AE channels can be monitored simultaneously. In addition, six parametric inputs are available.

The AEM is controlled by means of front-panel pushbuttons and a numeric keypad, which allow the operator to compose simple English sentence commands. Output is displayed on a 16-segment LED display, with hard copy provided on a built-in 40-column printer. Three modes of sensor-separation distance calibration are featured: (1) automatic via round-robin pulsing of each sensor channel, (2) manual via operator-controlled single-channel pulsing, and (3) operator via keypad numeric entry. Gain settings, distance calibrations, and flaw model parameters for the algorithms are stored in battery-backed RAM, which also allows storage of monitoring results for up to 999 tests and 1000 flaw indications.

The AEM can be interfaced to any IBM-compatible personal computer via its RS232 port to provide remote control of the AEM and recording of raw AE data in DOS-compatible files for post-test analysis. The recorded files may be played back into the AEM to allow model parameter optimization, or the data files may be examined and analyzed using any of the popular spreadsheet or database managers that run under DOS.

The AEM has been successfully field-proven in welding fabrication shops. It is easy to operate and allows welds to be repaired on a pass-by-pass basis, which eliminates full-section repair and excessive material handling. The AEM has also been demonstrated to be effective in detecting and locating growing cracks on steel highway bridges under normal traffic-loading conditions.

4.2.2 Vibration Analysis

Transverse-Impulse Vibration Analysis Technique. This technique, being developed at SWRI for NDE of ropes, involves applying a transverse-impulse force to a rope, detecting the resulting motion of the propagating impulse wave, and analyzing the detected signal)(20,(21). Figure 19 illustrates schematically the implementation of the technique. The impulsive force is applied by tapping the rope near one of the terminations using a small rod or hammer. The resulting wave motion is detected with a laser-displacement or electromagnetic vibration sensor. The signals are digitized and then are displayed, analyzed, and stored.

The technique can detect localized defects (such as broken strands and corrosion in a rope), which reflect the propagating impulse wave and, thus, produce defect signals. Figure 20 shows an example of data taken from a rope containing a localized defect. The technique also can be used to determine the tension in the rope from the transit time of the wave. The technique allows the inspection of the entire free length of the rope between two anchoring (or termination) points from a single sensor location. However, the technique can only inspect the free span between the anchoring points and, therefore, is not suitable for inspection of terminations.

On a stiff rope, the wave exhibits dispersive characteristics; that is, the wave propagates with different velocity depending on its frequency (a higher frequency component travels faster) (22). Because of dispersion, the initially narrow wave packet becomes broader with time. This broadening of the wave packet can interfere with the reflected signal from a localized defect and, thus, can make the defect detection more difficult (23).

However, these dispersion characteristics can be used to determine a parameter directly meaningful to structural engineers. The stiffness of a rope, which governs the degree of wave dispersion, decreases with the deterioration of the rope through development of defects such as broken wires and corrosion (24). The decrease in stiffness reduces the degree of wave dispersion. Therefore, by measuring the wave dispersion characteristics using the transverse-impulse vibration technique, the degree of rope deterioration can be directly assessed (25).

This technique has potential for a quick survey of the conditions of suspender ropes in suspension bridges.

4.3 Residual Stress Measurement

In another area of steel bridge inspection, FHWA recognized the need for instrumentation that would nondestructively measure residual stresses in bridge components. A research contract was awarded to SWRI to investigate and develop a nondestructive system that not only would quantitatively measure the surface stresses but also internal stresses in the component under test (26). After an in-depth assessment of 26 candidate methods, the Barkhausen Noise Analysis method for indication of surface stresses and the ultrasonic shear-wave birefringence for indicating average bulk stresses were selected for further research. Subsequently, this led to the development of portable prototype instruments for each of the two methods. The results of limited field tests utilizing the prototypes indicated that the two methods were useful in establishing baseline information; however, considerable research needed to be done addressing the anisotropy of specific materials and its effect on residual stress measurements. Subsequently, work was begun with the Magnetically Induced Velocity Change technique discussed later to overcome problems with both Barkhausen and shear-wave birefringence.

Barkhausen Noise Analysis. Under a time-varying magnetic field, the magnetic flux density in a ferromagnetic material does not change in a strictly continuous way, but rather by small, abrupt, discontinuous increments called Barkhausen jumps (Figure 21). With the Barkhausen Noise Analysis (BNA) technique, a time-varying magnetic field is applied to the specimen using an electromagnet; and the resulting Barkhausen jumps typically are detected by using an inductive coil. Figure 22 shows a schematic diagram of an instrument for sensing the Barkhausen effect. The detected signal, which is a burst of noise-like pulses (Figure 23), is filtered and processed.

Certain features of the processed signal (such as the maximum amplitude, root-mean-square amplitude, or applied magnetic field strength at which the maximum amplitude occurs) are used to determine the stress state in the material. Figure 24 illustrates a typical stress dependence of the Barkhausen noise signal amplitude in steel with positive magnetostriction. When the applied magnetic field is parallel to stress direction, the signal amplitude increases with tension and decreases with compression; the opposite occurs when the field is perpendicular to the stress direction.

Due to the eddy current screening, the inductively detected Barkhausen noise reflects the activity occurring to a depth of approximately a few thousandths of an inch. Thus, the inductive BNA technique is suitable for measuring only near-surface stresses. Measurements can be made, however, within a few seconds; and continuous measurements at a scanning speed up to a few feet per second are possible. Also, BNA does not require the preparation of the surface of a part under testing unless a thick coating or rust is present. Portable, field-usable Barkhausen instruments are now available commercially.

Results of Barkhausen noise measurements are sensitive to factors not related to stress such as microstructure, heat treatment, and material variations. Careful calibration of instruments and data analyses are essential for reliable stress measurements. A recent approach utilizing stress-induced anisotropy in the Barkhausen noise has significantly reduced the effects of the stress-unrelated factors and thus has improved the accuracy of the technique to approximately ± 5 ksi. The anisotropy is defined as $(A-B)/(A+B)/2$ where A and B are the signal amplitudes obtained with the applied magnetic field parallel and perpendicular to stress direction, respectively. This approach, however, is effective only for measurements of uniaxial or quasi-uniaxial stresses.

The stress-measurement range of the technique is effective up to about 50 percent of the yield stress of the material because the change in the Barkhausen noise with stress becomes saturated at these high stress levels. Consequently, the usefulness of the technique for residual welding stresses in bridge members may be limited.

Barkhausen jumps can also be detected by an acoustic emission sensor. The acquired Barkhausen noise can be used for stress measurements. Since the acoustic waves travel through materials, bulk stress can be determined by using the acoustic Barkhausen-noise technique. However, the technique cannot distinguish tension from compression, and its effective stress-measurement range is limited to about 50 percent of the yield stress. Practical application is presently hampered by the difficulty in differentiating acoustic Barkhausen noise from other noise produced from surrounding environments (as is the case with AE). The applicability of the acoustic Barkhausen noise to stress measurements in steel bridges has not been investigated yet.

Shear-Wave Birefringence. With shear-wave birefringence techniques, the difference in the velocities of two shear waves of mutually orthogonal polarization directions is measured and related to stress. Figure 25 illustrates the effect of uniaxial stress on the shear-wave velocity in steel. When the shear-wave polarization is parallel to the stress, the velocity increases under compression and decreases under tension. The opposite occurs when the polarization is perpendicular to the stress direction. Because of the texture-related elastic anisotropy, the two shear velocities at zero stress are generally different. The stress dependence of the shear-wave birefringence, which is defined as the difference divided by the average of the two velocities, is shown in Figure 26. For biaxial stress conditions, the birefringence is proportional to the difference in the two principal stresses. As indicated by the figure, to determine the stress, the texture-induced birefringence must be known. FHWA funded work with this approach at SWRI in the mid-1970s (26), but the texture-related problems prevented its application.

Magnetically Induced Velocity Change Measurements. The elastic moduli of a ferromagnetic material change when a magnetic field is applied to the material. This phenomenon, called the ΔE effect, has been known for more than 100 years. The ΔE effect can be measured by sensing changes in the ultrasonic-wave velocity in the material. The magnetically induced velocity change (MIVC) technique for stress measurements, developed originally at SWRI in the early 1980s under an internal research project, relies on the stress dependence of this magnetically induced ultrasonic-wave velocity change (27). The detailed stress dependence of the MIVC varies, depending on the ultrasonic wave mode used (longitudinal or shear), relative orientation between the stress axis and the magnetization direction, and material grade (28),(29),(30). However, to give a general idea on the effect of stress on the MIVC, its qualitative behavior is illustrated in Figure 27. The tensile stress reduces the amplitude of the MIVC whereas the compressive stress not only reduces the amplitude but alters the shape of the MIVC. The magnitude of the stress effect is in proportion to the stress values (no saturation with stress).

The instrumentation used for measuring MIVC is illustrated schematically in Figure 28. The ultrasonic waves are transmitted into the material using an ultrasonic transducer coupled on the material's surface. The reflected ultrasonic waves are detected by using the pulse-echo or pitch-catch technique. Magnetic fields are applied to the material using an electromagnet, and the applied magnetic field strength is monitored using a Hall-effect probe. The MIVC is detected by sensing the changes in the transit time of ultrasonic waves. Since the MIVC is small (on the order of 0.1 percent), a special ultrasonic instrument is used to detect the MIVC accurately. The MIVC is then measured as a function of the applied magnetic field.

In a project for FHWA conducted in 1983-1985 (31), the capability of the MIVC technique for measuring residual welding stress through the thickness (0.5 inch) of a steel plate was demonstrated at SWRI. In this project, a prototype instrumentation system for MIVC measurement was developed and delivered to FHWA for use in laboratory.

The development of the technique and the system continued at SWRI. The system, now fully computerized, reads in the transit time change, converts it to MIVC, and displays and stores the data automatically. The technique and system have been applied to measurements of biaxial stresses in 48-inch-diameter, 0.5-inch-thick pipes. With the combined use of both longitudinal and shear wave MIVCs, measurements of the magnitude and sign (tension or compression) of each of the two principal stresses can now be made.

This technique can be used to measure the stress state in steel bridge members to allow assessment of the severity of a defect (detected by some other techniques) and determination of the structural integrity of the member based on fracture mechanics analysis.

4.4 Signal Processing, Analysis, and Display

The FHWA UT system, or UBIS, discussed previously, combines the power and flexibility of the latest computer-imaging techniques with the simplicity of a portable unit (7). The system is a compromise of the twofold problem of detection and operational needs. It provides excellent record-keeping capabilities, various post-analysis options, and reasonable flexibility in order to avoid using several types of UT instruments that would have required substantially more time and effort to deploy.

The UBIS combines A-scanning and holography into a three-tiered approach for inspecting bridges. The first level is the fairly quick A-scan for defect detection. The second level is the automated imaging of the defect using B-scan, C-scan, and time-of-flight displays. The final level is the use of holographic imaging and reconstruction which may require a second scan using an optimized transducer. In the first level, the downhole operator moves the transducer over the surface of the inspection area while observing the A-scan display. When the A-scan indicates the need for further documentation, the downhole operator positions the hand scanner where it is needed to cover the small region in question and within the motion limits of the hand scanner. The operator then inserts the proper transducer in the fixture at the end of the arm. The system is supplied with 1-, 2.25-, and 5-MHz transducers and with 45-, 60-, and 70-degree angle-beam shoes. The choice of the angle for surface or near-surface cracks is generally 60 or 70 degrees. For deeper crack-like defects, a 45- or 60-degree angle-beam shoe is used. Once the transducer is selected and attached to the scanner arm, the operator positions it at the scan aperture origin and signals the uphole operator.

Five major features of the data are displayed on the computer monitor (Figures 29 and 30). The large window (upper left) is the top or plan view (C-scan). The windows at the side and below the plan view window are the column and row (X-Y) profiles of the data (B-scans). The last two features shown are the color-coded display (far right), showing the maximum value of a particular color, and the X-Y coordinates in inches (lower center) along with the amplitude value at that point. Time-of-flight data are displayed in much the same way.

The key steps in this transformation are digitizing the analog ultrasonic waveforms at very high speeds, rapidly transferring these digitized data into computer memory's executing algorithms to transform the raw UT data into an image of the interior of the object, and then transferring this image from the computer to an appropriate display device. The desired input test parameters are set by the operator and include the peak amplitude in the gate, the time of flight within the gate to the first signal above a preset threshold, and the sine and cosine coefficients that are used to reconstruct a holographic image.

The holographic reconstruction method is only used when a detected defect's size exceeds a prescribed safety value. The procedure is time-consuming and should be used conservatively. For more information on the holographic techniques used by the FHWA system, see the "Automated Imaging System for Bridge Inspection" (FHWA-RD-87-068).

5. RESEARCH AND DEVELOPMENT NEEDS

In the past, NDE R&D needs have often been expressed in qualitative terms such as the need for improved safety; cost-effective maintenance inspections; inexpensive systems; and systems that do not require highly specialized training or highly paid personnel to operate them, cause minimal traffic disruptions, are able to detect hidden damage such as corrosion in inaccessible areas, and--perhaps above all--give early warning of imminent bridge collapse. Discussions have even been held on creating a special "Inspection Engineering" discipline to address the complexity of the bridge inspection problem. All of these are good ideas, but the challenge of this conference and workshop was to try to put some "meat on the bones" and to establish a consensus on what the participants believe to be the rank-order priority for each area targeted for R&D.

After reviewing the state of the art in NDE of steel bridges, the fundamental problem appears to center on knowing *where and how to look.* There are thousands of *miles* of welded steel bridge girders to inspect, not including the miles of steel superstructures and cables on suspension bridges! Valuable resources that are in short supply should not be spent looking in detail at sound structure. Therefore, structural monitoring and NDE methods and techniques are needed to determine where and how to concentrate increasingly scarce inspection and maintenance resources and, most importantly, that give bridge owners and the public early warning that a significant safety problem exists.

In addition to the problem of knowing where and how to look, access to the area of concern is often difficult. Further, the insidious nature of slow deterioration due to corrosion (like a growing cancer) is a mushrooming problem on many of our older and aging bridges. So then, what are the NDE R&D needs and what are their priorities? This conference and workshop attempted to answer these questions for steel and other types of highway bridges.

The following sections present a brief summary of the accomplishments with respect to NDE of steel bridges and further insights into NDE problems and needs by the structural engineering community as articulated by Professor Karl Frank of The University of Texas at Austin.

5.1 Conference Summary

This section presents a brief summary of the steel bridge portion of the FHWA Conference on NDE of Bridges held August 25-27, 1992, in Arlington, Virginia. On the first day of the conference, presentations were made by Professor Karl Frank on NDE Problems from the structural engineering perspective and by the author of this report, Dr. Cecil Teller, on the State of the Art in NDE of Steel Bridges from the NDE research and development perspective.

A workshop session was held on the second day with those attendees who were particularly interested in steel bridges representing:

- State Highway Departments
- Federal Highway Administration
- NDT Equipment Companies
- Researchers
  - Universities
  - Federal Laboratories
  - Small Businesses
  - Not-for-Profit Institutes
- Practitioners
- Others

On the morning of the second day of the conference, the workshop on steel bridges began with informal presentations by several participants. Information on a variety of NDE methods was presented (see the Appendix). Anyone who was interested in making a presentation was given the opportunity. (Also, anyone who had additional information for this paper was asked to send it to the author within a couple of weeks; no one responded, however.) In the afternoon, a "strawman" for NDE R&D technology needs (see Figure 31) was used to initiate discussions, and a general "brainstorming" session was held to surface issues that the participants felt were relevant to the workshop objective of setting R&D needs and priorities for NDE of steel bridges (see Figure 32). This brainstorming session was wide open, and everyone who wanted to contribute something was allowed to do so.

After all the issues were listed that the participants could think of, those participants representing the bridge owners and inspectors got together with Professor Frank and came up with a list of priority needs for NDE R&D mainly in two areas: (1) in-service bridges and (2) fabrication inspection (see Figure 33). It is these lists that were the main contribution of the workshop participants to the objectives of the conference to:

> Exchange information via white paper presentations, workshop sessions, and informal discussions in order to prepare a plan giving specific guidance for studies on NDE of bridges proposed as a part of the High-Priority National Program Area. Identify the most feasible and practical NDE methods to be developed and used for bridge inspection.

Lastly, on the morning of the third day, a summary of the steel bridge workshop findings was presented by Professor Frank and Dr. Teller. The presentation of this summary concluded the conference and workshop.

5.2 Further Insights into NDE Problems and Needs

In a paper (32) covering his presentation to the conference, Professor Frank stated that:

> New technology, improvement to existing technology, or automation are needed to provide more reliable and less costly structures. Coupled with improvements in inspection technology, new acceptance criteria must be developed which allow the improvements in inspection to be translated into more rational acceptance and evaluation procedures. The inspection equipment and procedures must be capable of producing quantitative information on flaw size and location. Technologies that require subjective evaluations are not reliable enough to insure the safety of our bridges.

Professor Frank concluded his paper by saying that steel bridge NDE research programs should emphasize:

(1) Rapid and automated inspection techniques,

(2) Techniques which provide quantitative flaw size information that is reliable and is not subject to subjective operator interpretation, and

(3) Integration of the sensitivity of the inspection methods and the severity of the discontinuity into realistic acceptance criteria.

It is very important for the NDE R&D community to heed these remarks since they come from the "customer," i.e., structural engineers and bridge owners.

6. CONCLUDING REMARKS

The stated goals for FHWA NDE R&D are:

- Improved bridge safety for the traveling public
- Reduced "downtime" due to bridge loss
- Reduced maintenance costs through early detection of structural problems

To achieve these goals, NDE has been identified as a High Priority National Program Area (HPNPA) with the following objectives:

- Improve existing methods of NDE for QC
- Improve local NDE used for identifying defects
- Develop reliable global NDE techniques

Significant new funding is available to achieve these goals and objectives under the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991. With the resources in hand *and* priorities set for FHWA NDE R&D (with the aid of the participants at this conference and workshop, especially the customers, i.e., the bridge owners who are responsible for bridge inspection and maintenance to ensure safety), all the ingredients are in place to move forward aggressively to provide improvements to and new designs for the highway transportation infrastructure well into the 21st century.

7. ACKNOWLEDGMENT

I am deeply indebted to personal contacts with practitioners and researchers in the NDE community for providing information for this paper and to the literature sources cited in the bibliography. Especially useful have been the FHWA's 1986 Training Manual on "Nondestructive Testing Methods for Steel Bridges" (1) and the FHWA's 1989 Report on "Advanced Bridge Inspection Methods: Applications and Guidelines" (2) from which I

learned a great deal and borrowed heavily in preparing this paper. I highly recommend that these documents be read by all who are interested in learning more about inspection practices on steel and other types of bridges. Any oversight of ongoing work is deeply regretted and most assuredly unintentional. Please accept my apologies in advance if such an oversight has occurred.

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FIRST CHOICE

SECOND CHOICE (shaded)

THIS CHART IS INTENDED TO BE USED AS A GUIDE IN SELECTING AN APPROPRIATE NDT METHOD. IT MUST BE USED IN CONJUNCTION WITH THE SPECIFIED NOTES.

		BASE METAL						FILLET WELDS						GROOVE WELDS							WELD-RELATED CRACKING			IN-SERVICE CRACKING
		STRINGER	SEAM	LAMINATION	CRACKS	CORROSION	NICKS/GOUGES	INSUFFICIENT THROAT	CONVEXITY	UNDERCUT	OVERLAP	INSUFFICIENT LEG	INC PENETRATION/FUSION	UNDERFILL/CONVEX	UNDERCUT	OVERLAP	LACK OF FUSION	INCOMPLETE PENETRATION	POROSITY	INCLUSIONS	FILLET WELDS	GROOVE WELDS	LAMELLAR TEARS	FATIGUE CRACKS
VISUAL		O	R	O	O	R	R	R	R	R	R	R		R	R	R	O		O	O	O	O	O	O^{1}
LIQUID PENETRANT	VISIBLE DYE	O^{2}	R^{2}	O^{2}	O^{2}												O^{2}		O^{2}	O^{2}	O^{2}	O^{2}	O^{2}	$O^{2,3}$
MAGNETIC PARTICLE	AC YOKE	O	R	O	O												O			O	O	O	O	O^{4}
	DC YOKE	O^{4}	R	O^{5}	O^{5}												O^{5}			O^{5}	O^{5}	O^{5}	O^{5}	$O^{4,5}$
	*PRODS	O^{5}	R	O^{5}	O^{5}												O^{5}			O^{5}	O^{5}	O^{5}	O^{5}	$O^{4,5}$
RADIOGRAPHY	X-RAY				S^{6}								S^{6}				S^{6}	S	S	S	S^{6}	S^{6}	$S^{6,7}$	$S^{6,7}$
	GAMMA				S^{6}								S^{6}				S^{6}	S	S	S	S^{6}	S^{6}	$S^{6,7}$	$S^{6,7}$
ULTRASONIC	STRAIGHT BEAM	S		S		R																	S^{9}	
	**ANGLE BEAM				S^{6}								S^{6}				S^{6}	S	S	S	S^{8}	S^{6}	$S^{6,7}$	$S^{6,7}$
ACOUSTIC EMISSION																		11	11	11	10	10	10	10

R - RECOMMENDED; S - RECOMMENDED FOR SUBSURFACE DISCONTINUITIES; O - RECOMMENDED FOR SURFACE DISCONTINUITIES; R^{1}, S^{1}, O^{1} - RECOMMENDED WITH RESTRICTIONS

* CAUTION IS ADVISED SINCE ARC STRIKES CAN RESULT FROM THE IMPROPER USE OF PRODS.

** SPECIALIZED ULTRASONIC TECHNIQUES FOR SIZING CAN BE APPLIED. THE USE OF THESE TECHNIQUES MAY INVOLVE CREEPING WAVE OR CRACK TIP DIFFRACTION. SPECIALIZED TRAINING NEEDED.

NOTES:

(1) The detectability of fatigue cracks will depend on size. Early stages may not be detectable. Magnification or additional NDT may be necessary.

(2) Surface preparation is critical. Discontinuities must be open to the surface.

(3) An increase in dwell time will enhance detectability, and the use of fluorescent penetrants will provide higher sensitivity.

(4) The wet method is preferred over the dry. For maximum sensitivity, fluorescent particles should be used. Surface preparation is critical. Nonrelevant indications (i.e., at weld toe) can mask indications.

(5) Although the direct current MT method produces subsurface flux in a material, the use of MT is recommended for surface discontinuities. RT or UT is preferred for subsurface.

(6) Discontinuities can be missed due to orientation.

(7) Detection capability is limited by size.

(8) Although not normally used for fillet welds, may be used.

(9) Some reported success when used for connected cracks.

(10) For detection, discontinuities must be actively growing.

(11) Some reported success in the detection of various weld discontinuities as they are being made.

Figure 1. Nondestructive Test Method Comparison Chart for Steel Bridges (1)

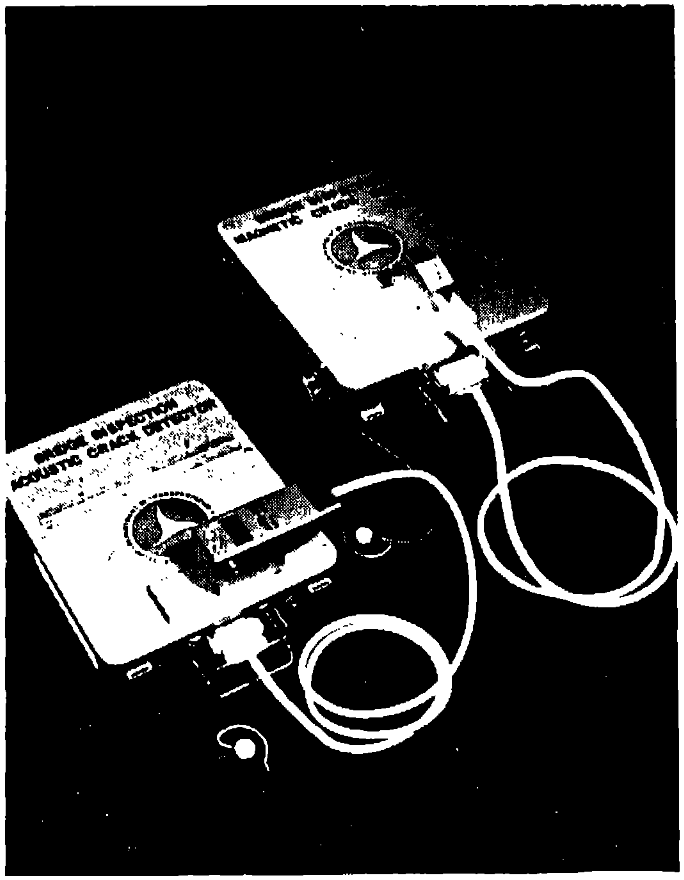


Figure 2. Acoustic Crack Detector/Magnetic Crack Definer (ACD/MCD) Steel Bridge Inspection System (5)

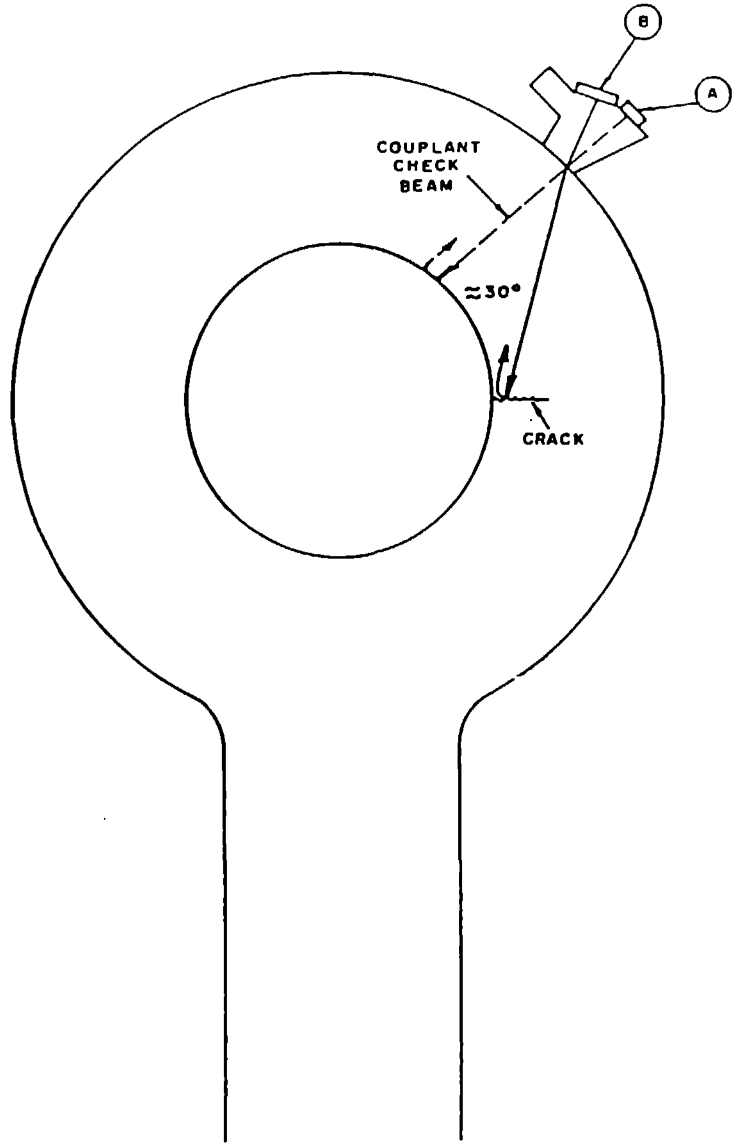


Figure 3. Schematic of Dual Transducer Periphery Probe. "A" is the Couplant or "Bore Channel; "B" is the Survey Channel. (5)

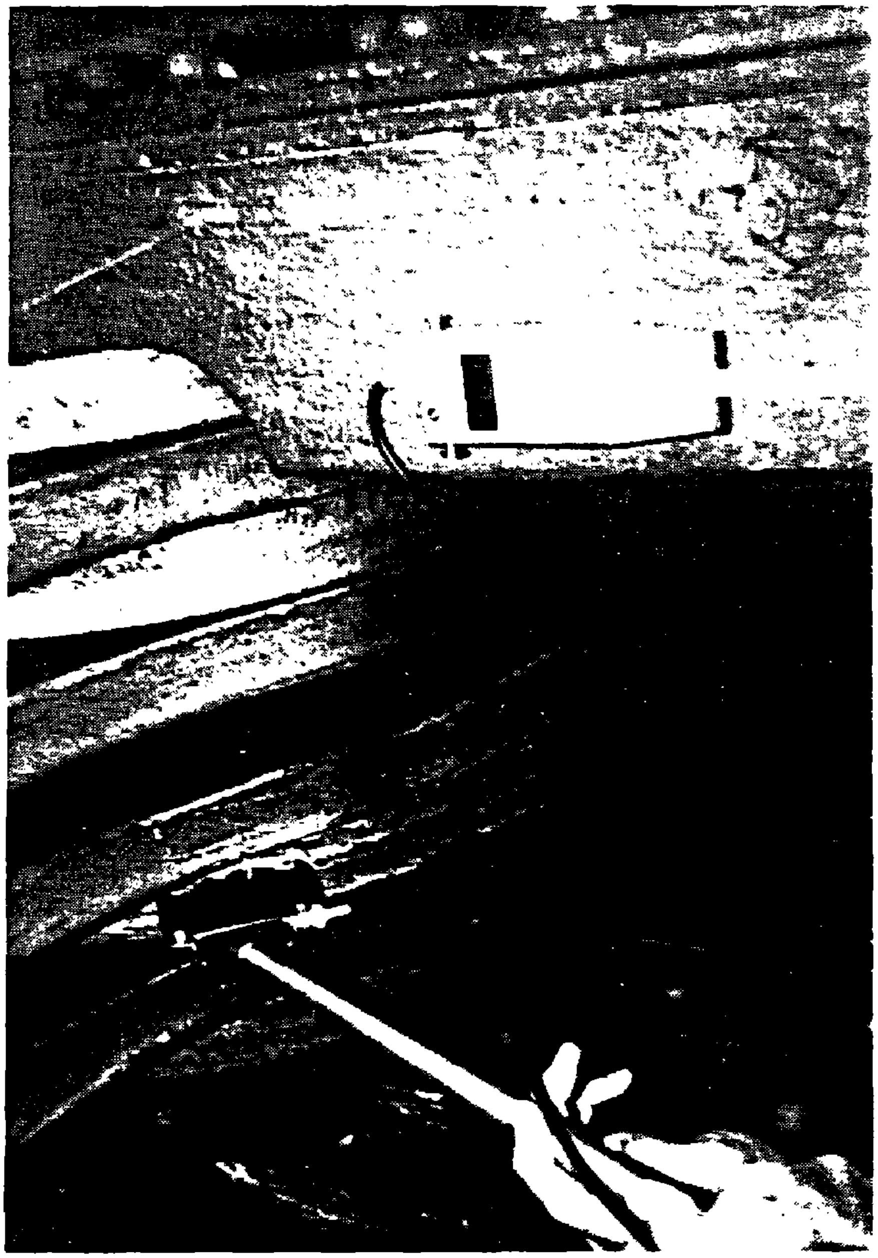


Figure 4. ACD Eye-Bar Periphery Probe (2)

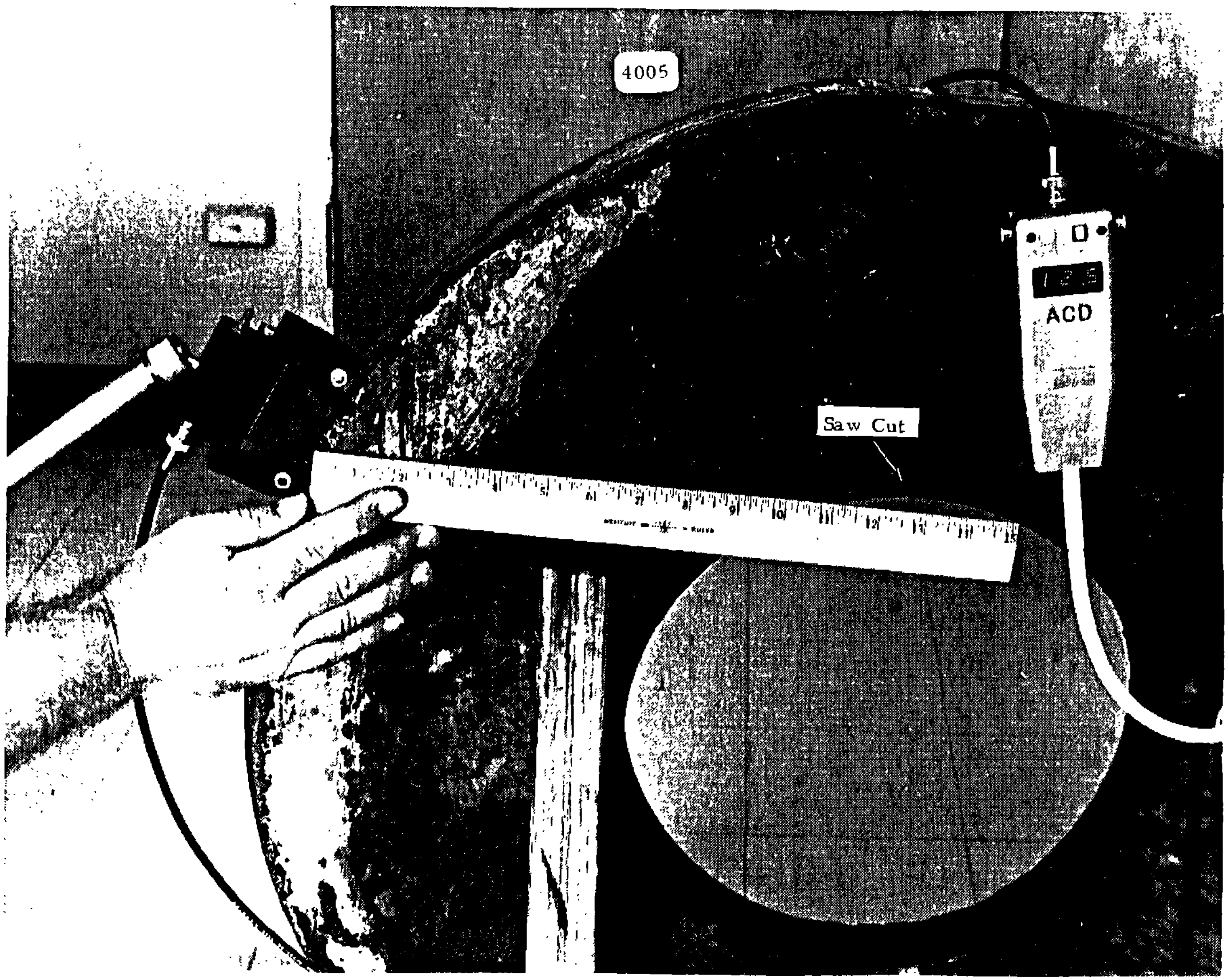


Figure 5. Crack (Notch) Signal of 1.31 Inches on Silver Bridge Eye-Bar Specimen (5)

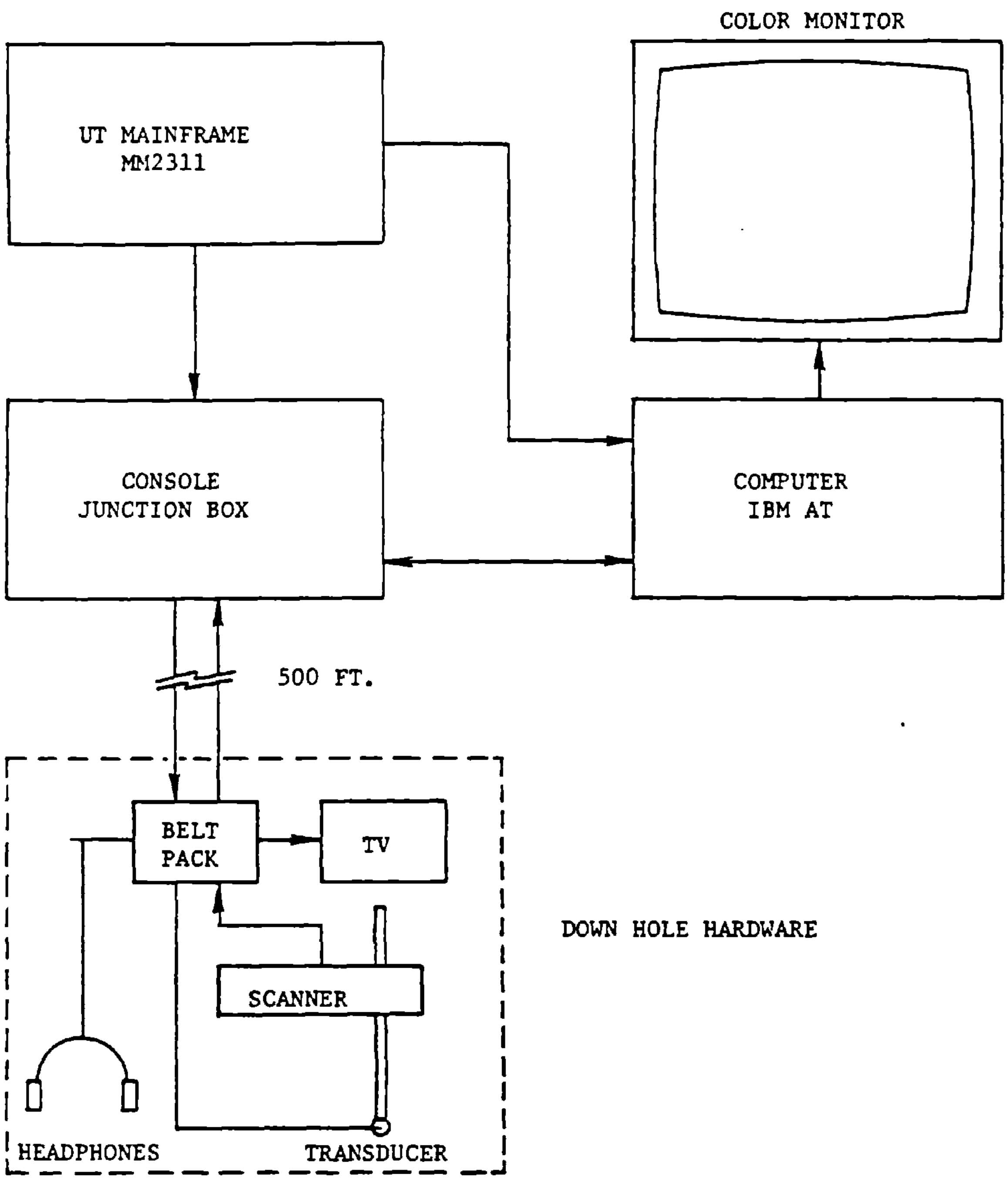


Figure 6. Uphole/Downhole Hardware of the Sigma-Developed UBIS for Bridge Inspections (2)

Figure 7. Operator Wearing the TV Pack, Belt Pack, and Scanner Transport Pack that Make up the Downhole Hardware (12)

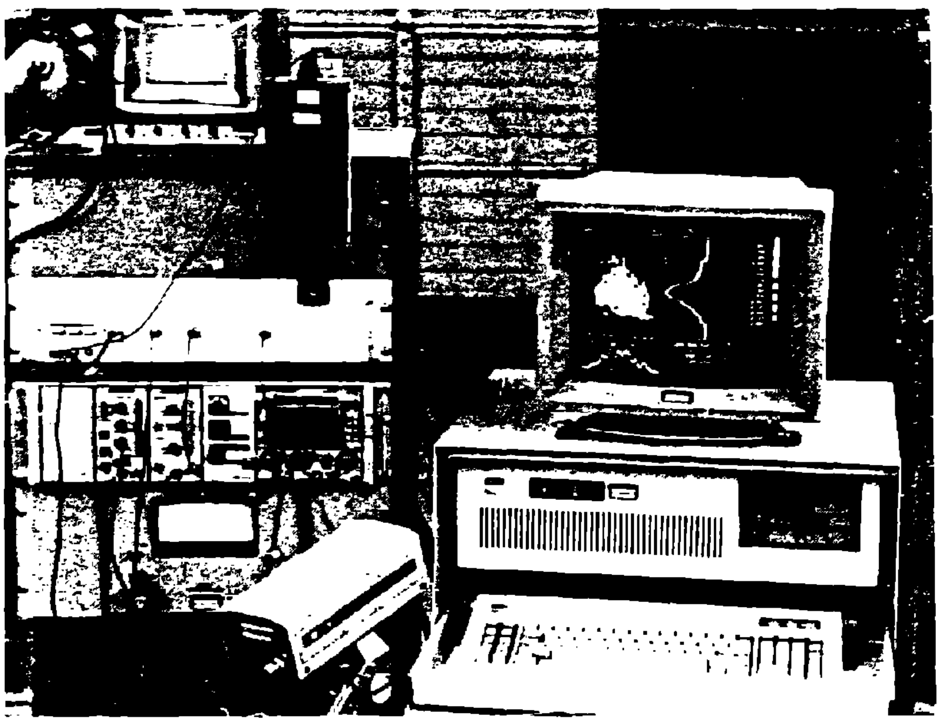


Figure 8. Uphole Hardware of the UBIS (2)

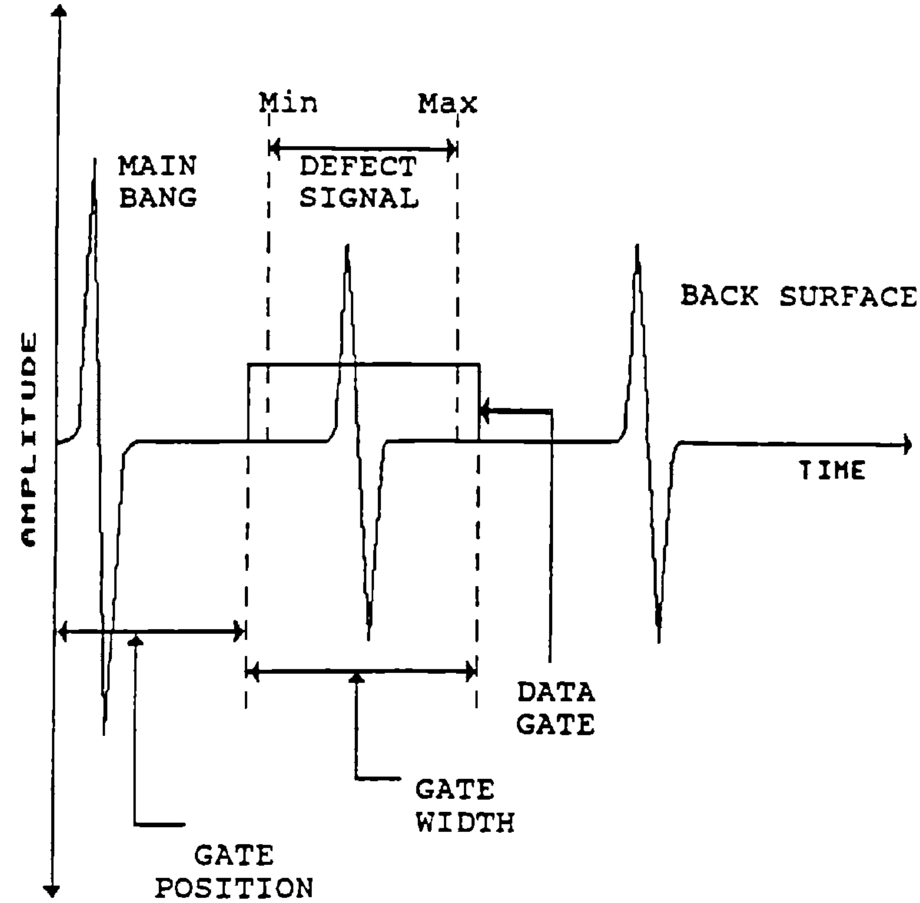


Figure 9. Data Gate Used in the FHWA System in Relation to the Signal Generated by a Returning UT Echo (2)

- The H-F UTV is worn in a small backpack and features a heads-up helmet-mounted virtual display.
- All functions are voice-activated from a throat microphone for hands-free use.
- The H-F UTV is IBM PC/AT compatible and supports standard formats for off-line postprocessing.
- Industry standard removable 3.5-inch disks store 450,000 C-scan grid points.
- The device is completely self-contained; no remote (uphole) operator is required.

Figure 10. H-F UTV Device from Sierra Matrix, Inc. (8)

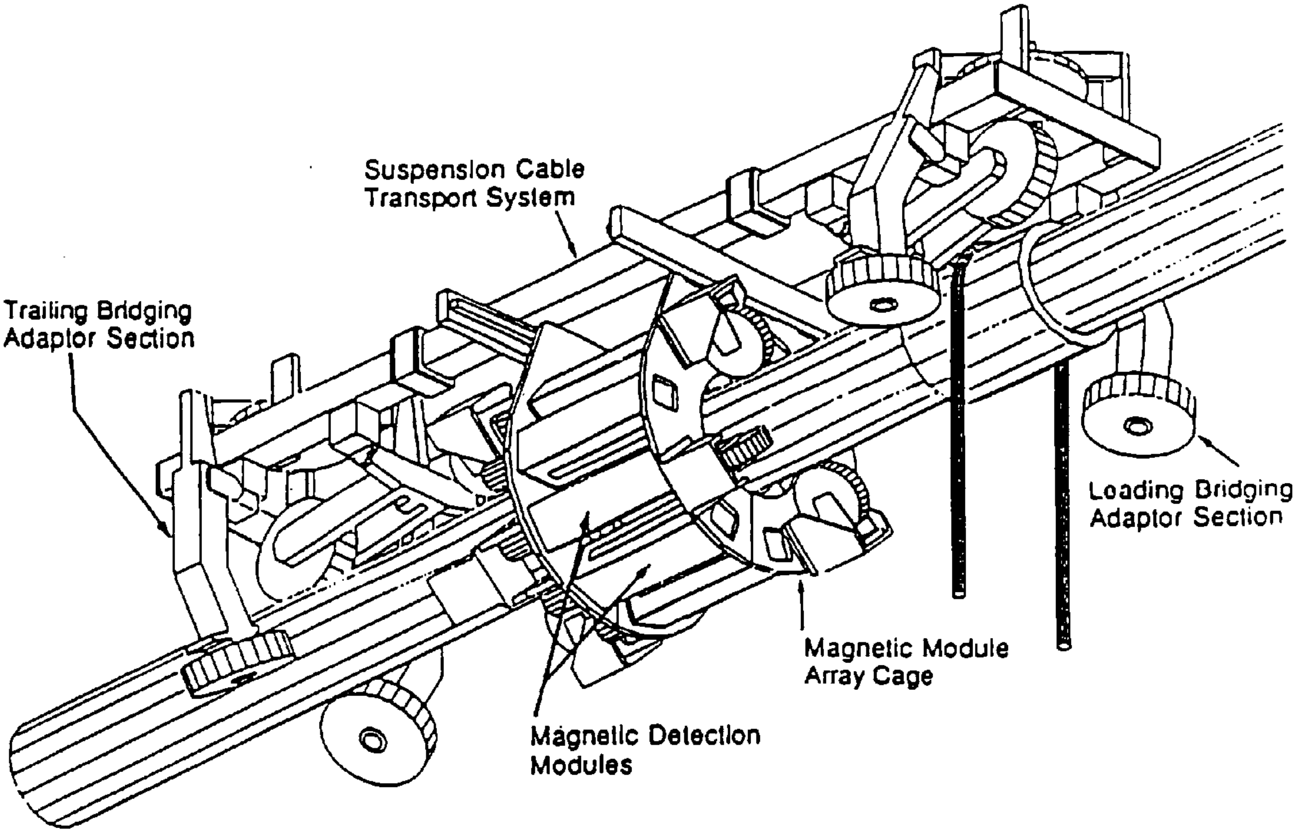


Figure 11. Magnetic Flux Leakage Inspection System (MFLIS) (12)

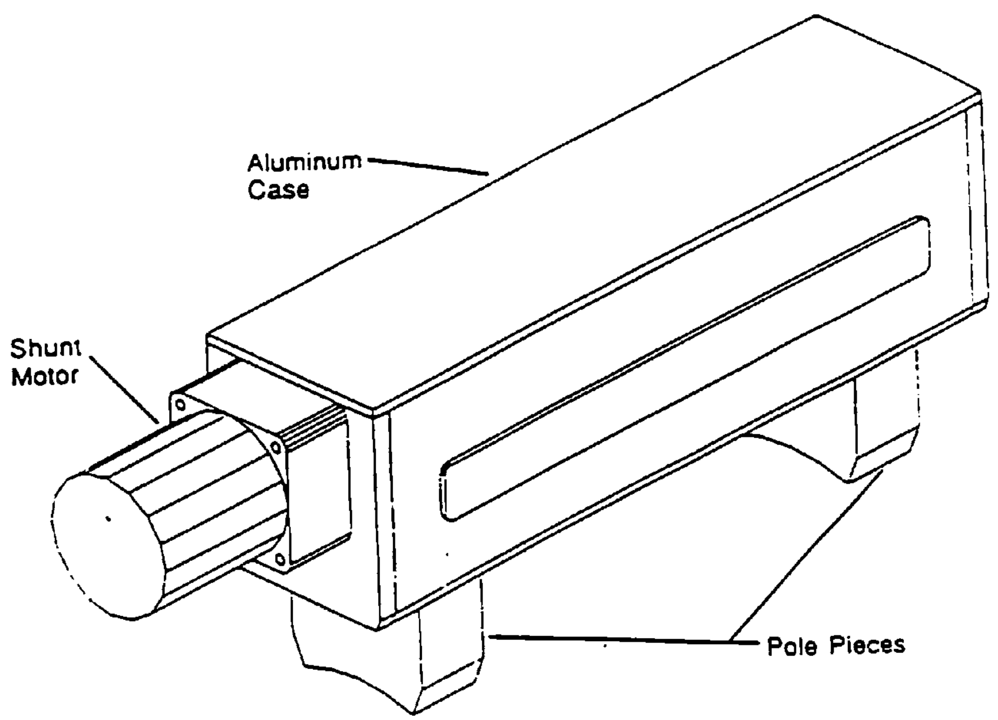
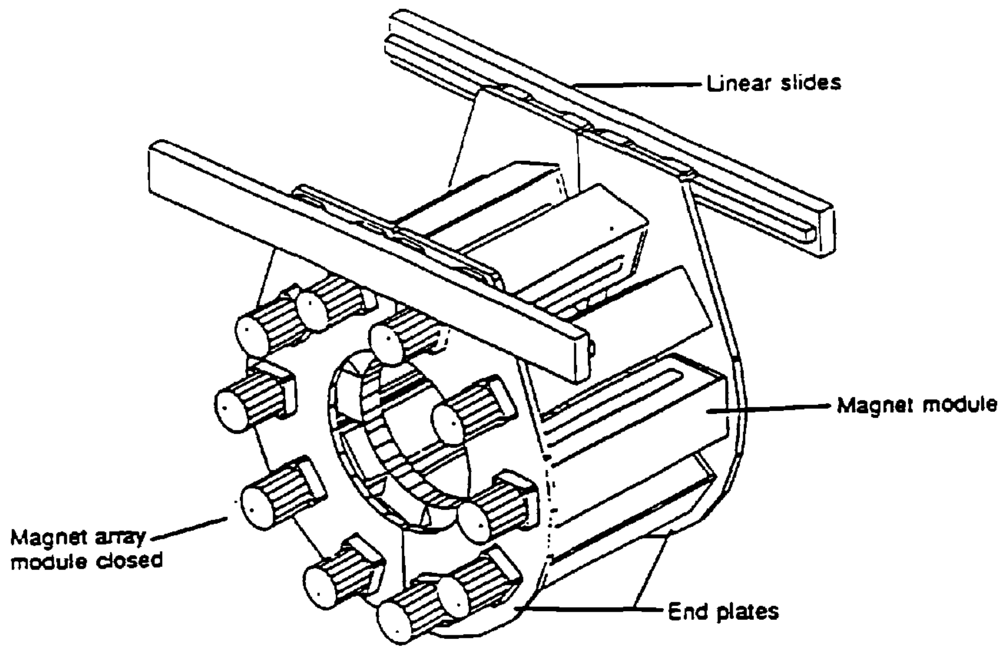
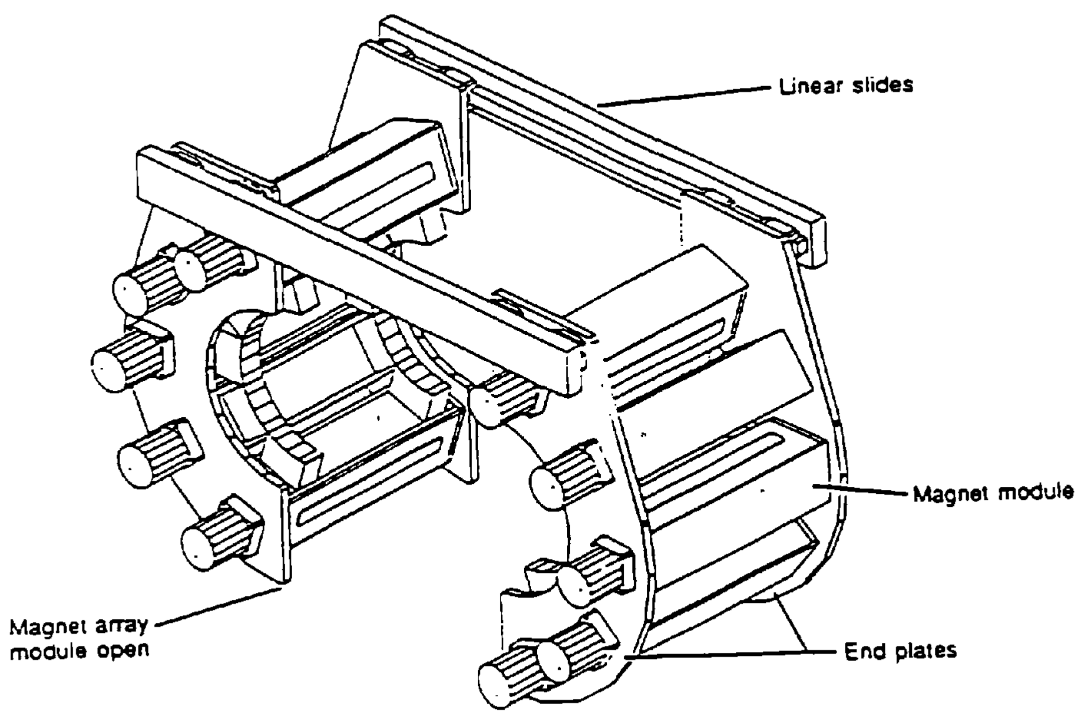


Figure 12. Top View of Complete Magnet Module (12)



a. Closed configuration



b. Open configuration

Figure 13 Magnet Array, 10 Module with End Plates, Linear Slides (12)

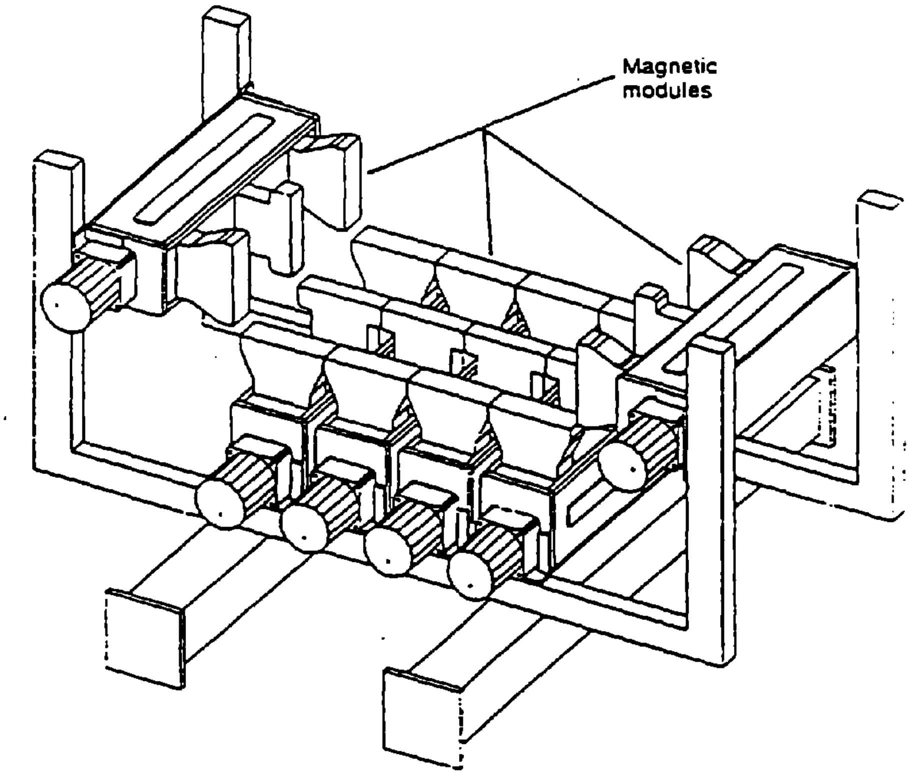


Figure 14. Multiple Magnetic Module Sensor Array for Beam Inspection (13)

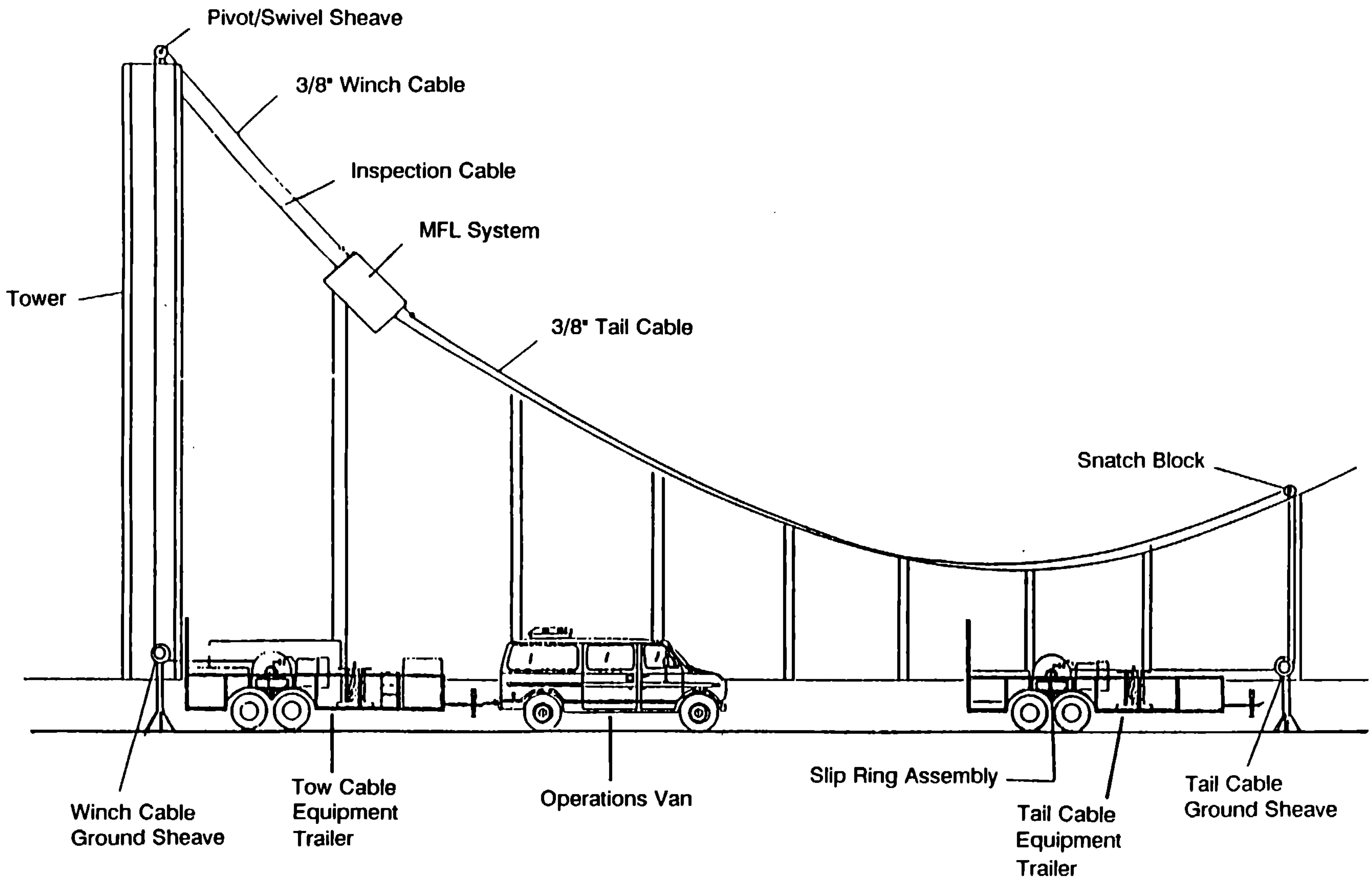


Figure 15. Operational Configuration for Suspension Cable Inspection (13)

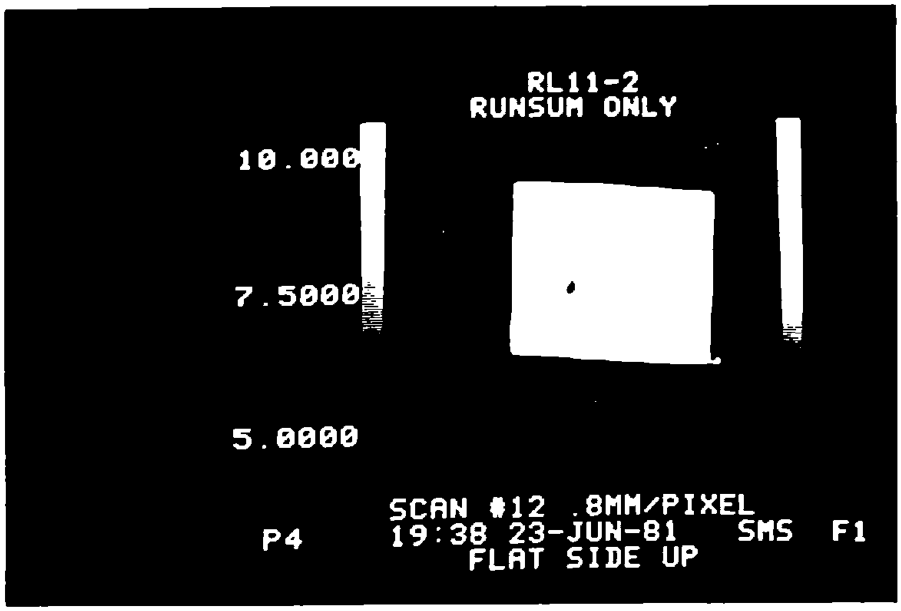


Figure 16. Tomogram of Butt Weld Showing a Black Triangular Void Centered in Interior (14)

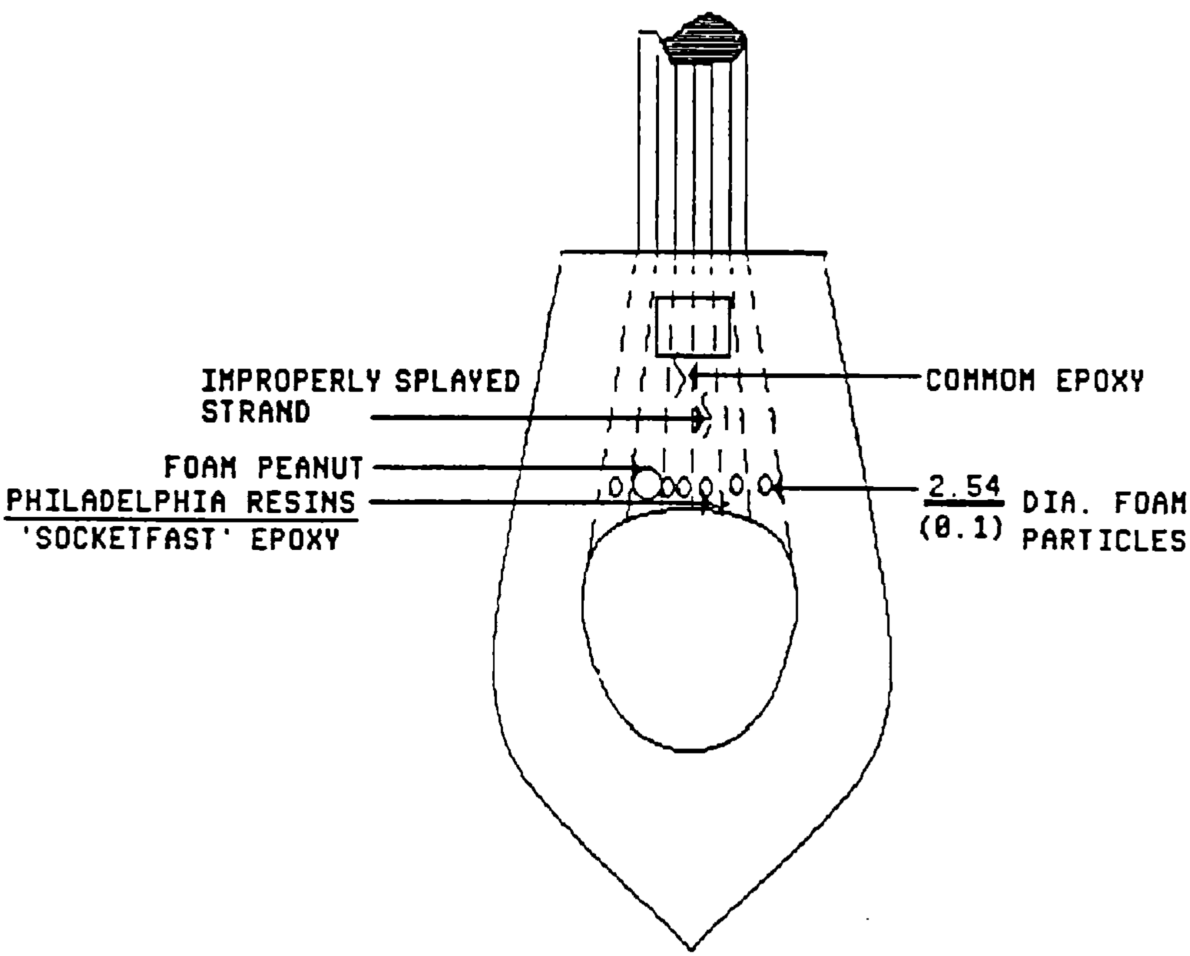


Figure 17. One of the Fittings Used in KTRP Test, with Artificially Induced Flaws Used as Defect Substitutes (2,15).

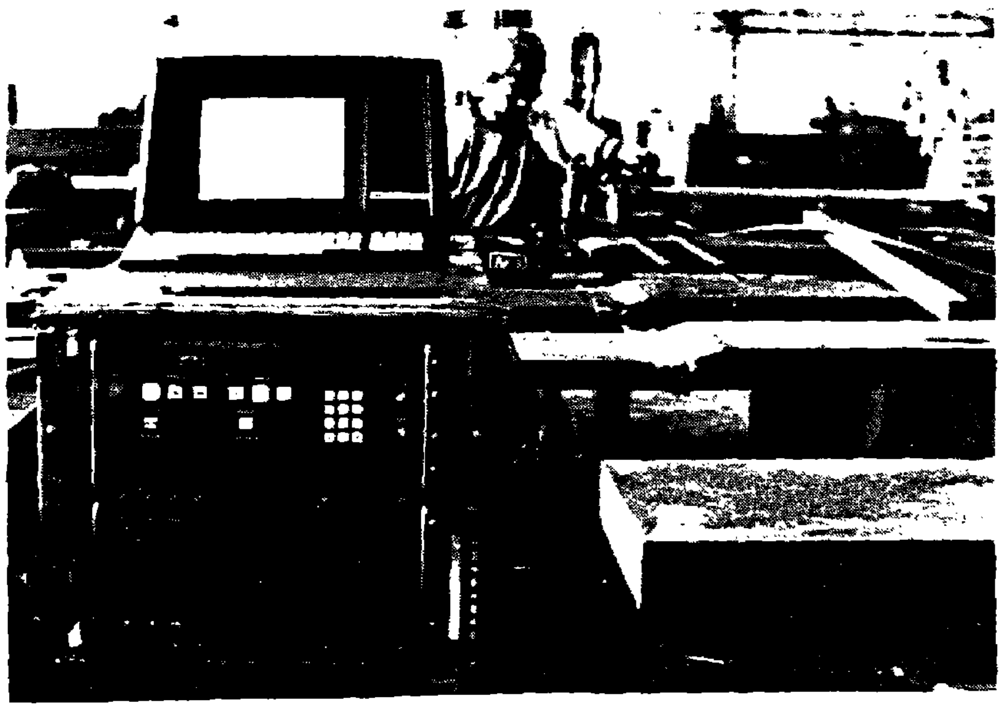


Figure 18. A Multichannel AE System with Computerized Signal Processing.
(This AEWM has also been used for in-service inspections of steel bridges (2,17).

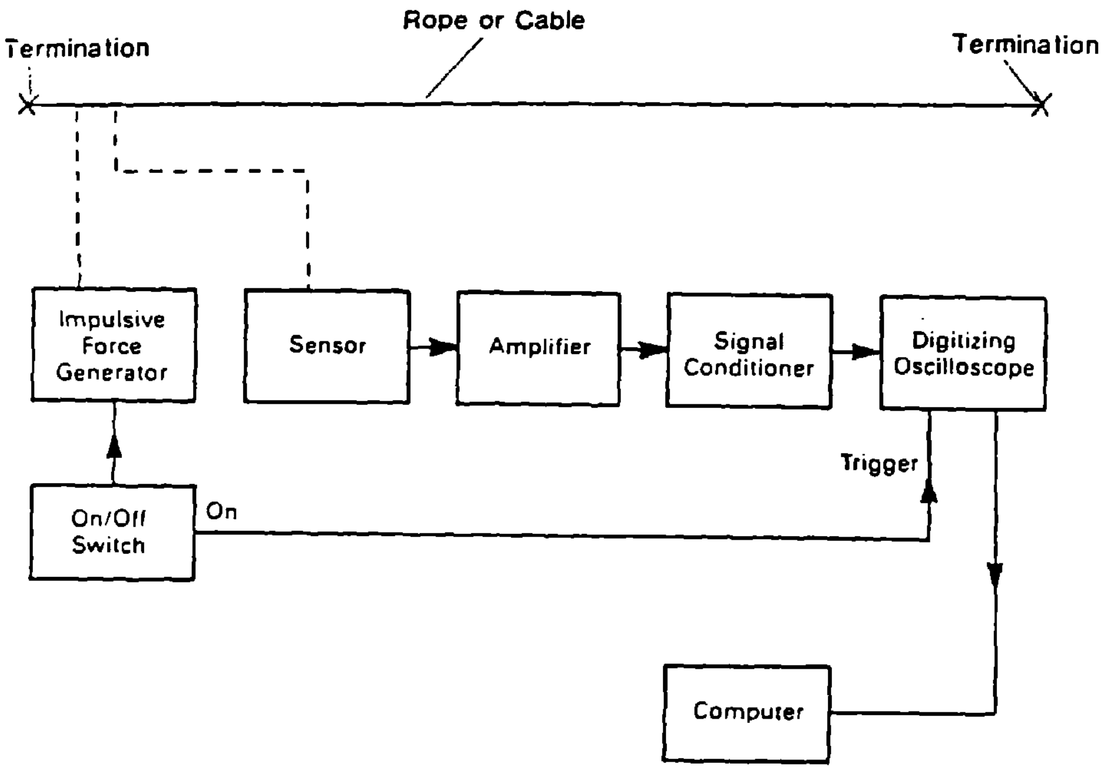


Figure 19. Implementation of the Transverse-Impulse Vibration Technique (23)

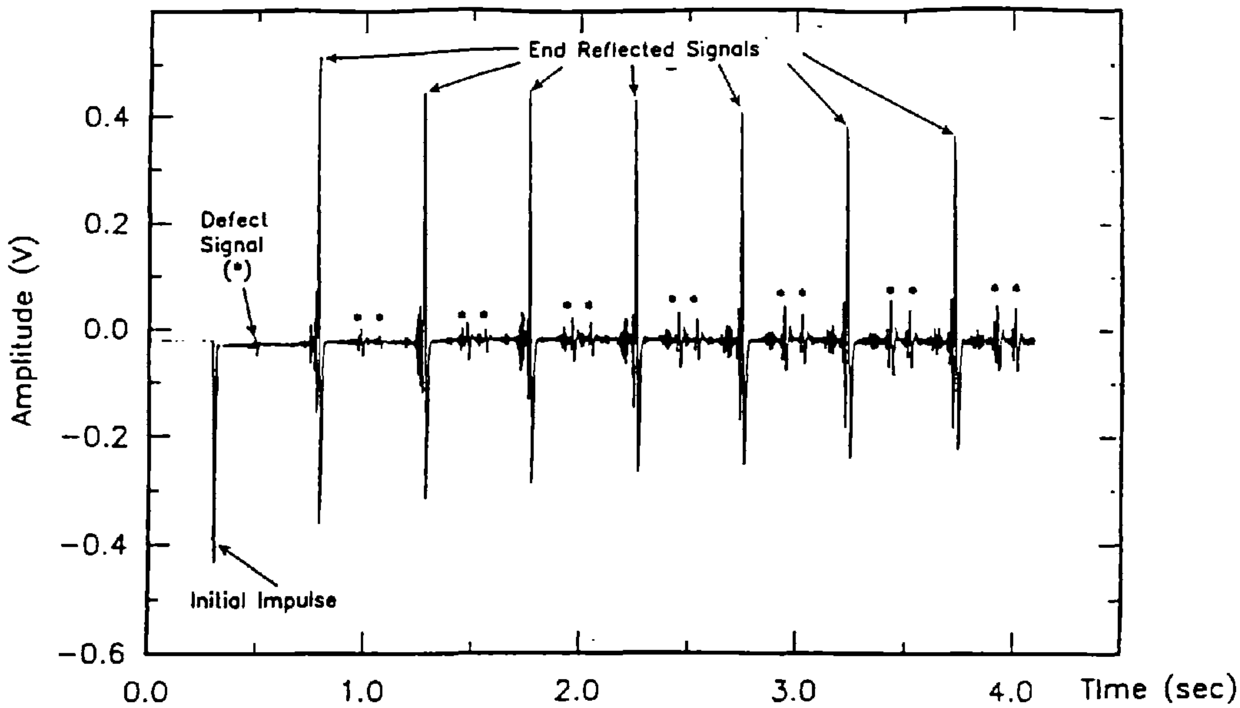


Figure 20. Example of Data Taken from a Rope Containing a Localized Defect (23)

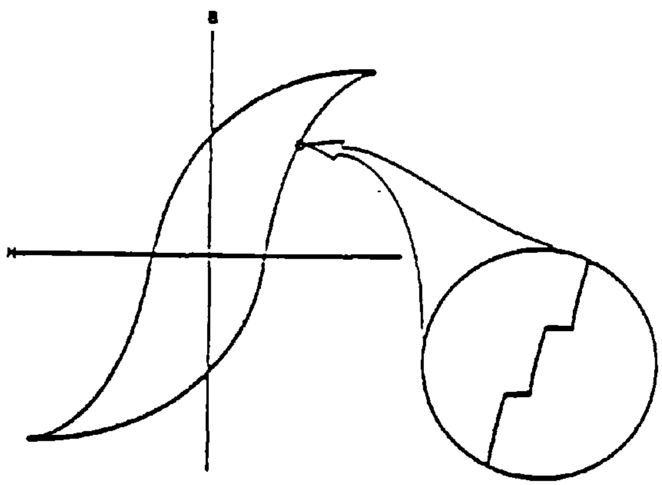


Figure 21. Hysteresis Loop for Magnetic Material Showing Discontinuities Producing Barkhausen Noise

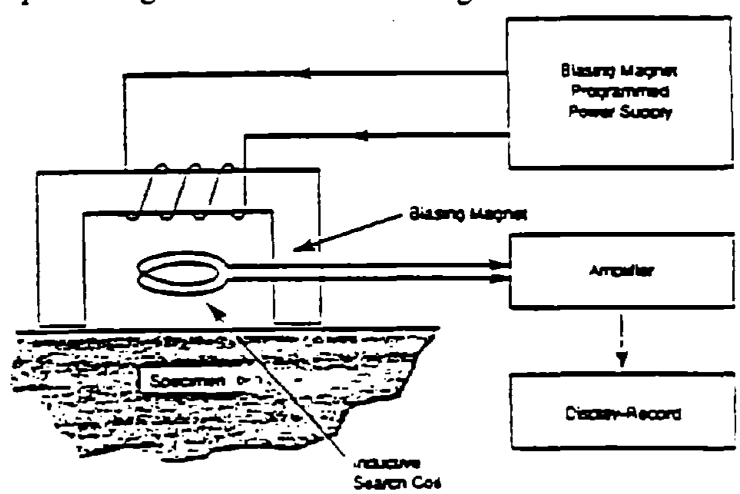


Figure 22. Arrangement for Sensing the Barkhausen Effect

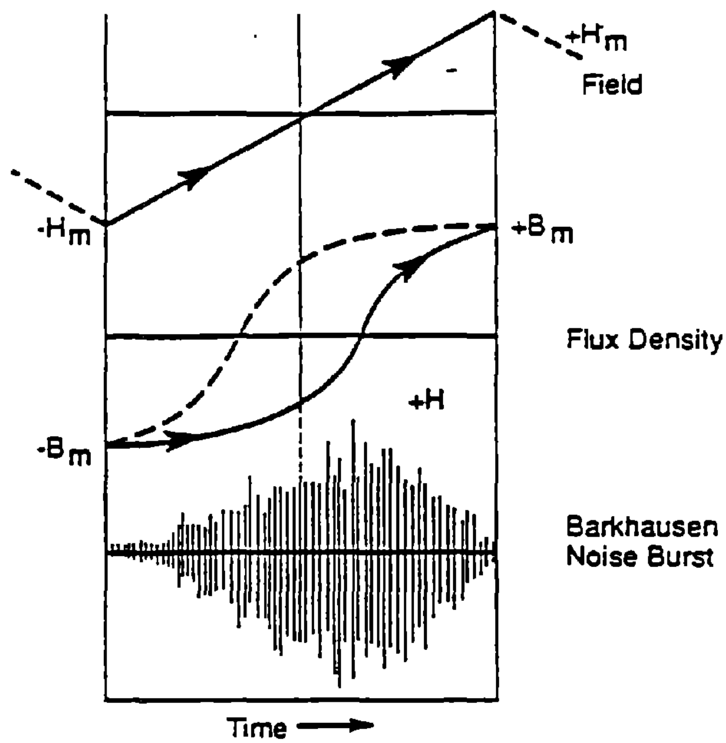


Figure 23. Time-Varying Applied Magnetic Field (H), Flux Density in the Material (B), and Barkhausen Noise Burst

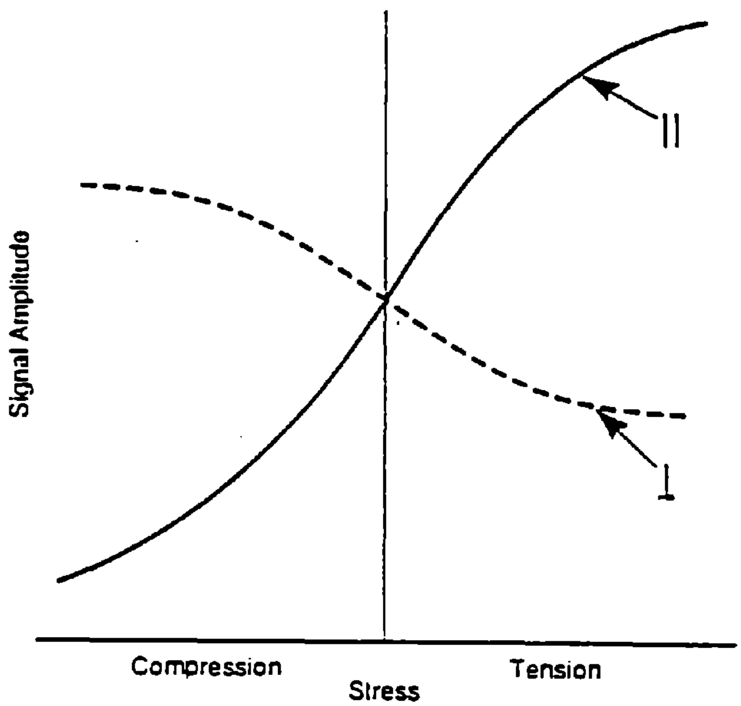


Figure 24. Stress Dependence of Barkhausen Noise with Applied Magnetic Field Parallel (||) and Perpendicular (⊥) to the Applied Stress Direction

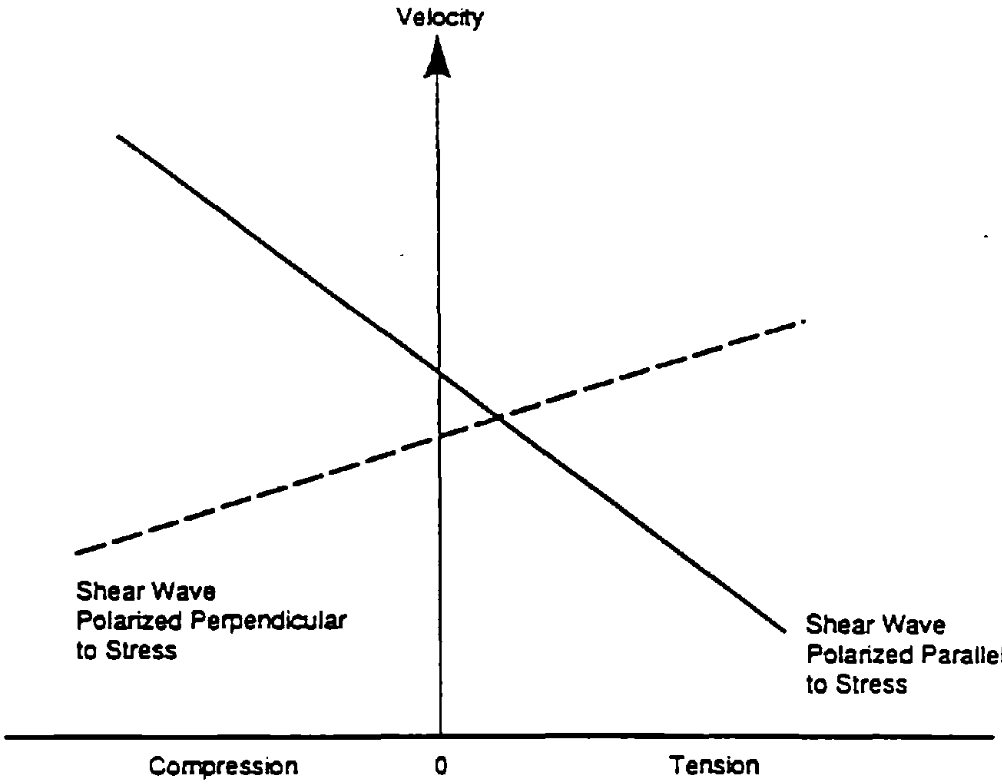


Figure 25. Effect of Uniaxial Stress on Ultrasonic Shear-Wave Velocity in Steel (28)

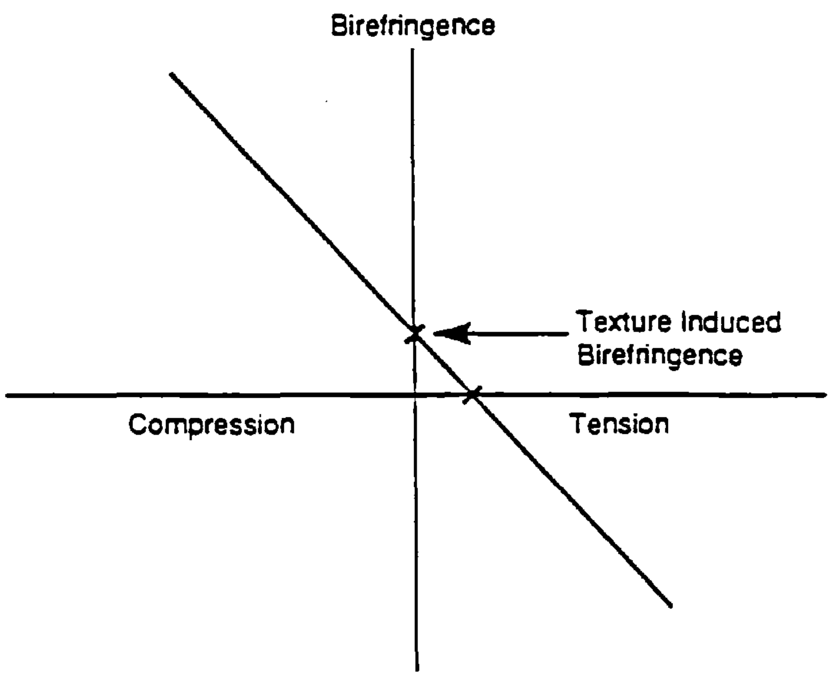


Figure 26. Stress Dependence of Shear-Wave Birefringence (28)

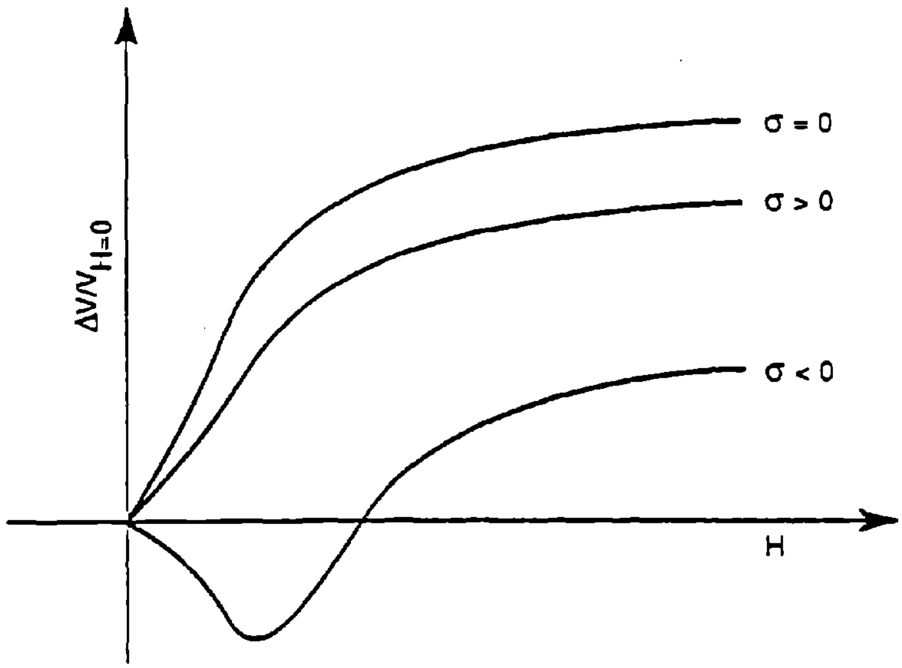


Figure 27. Change in Ultrasonic Velocity, ΔV, with Magnetic Field, H, under Various Stress Levels (28),(29),(30)

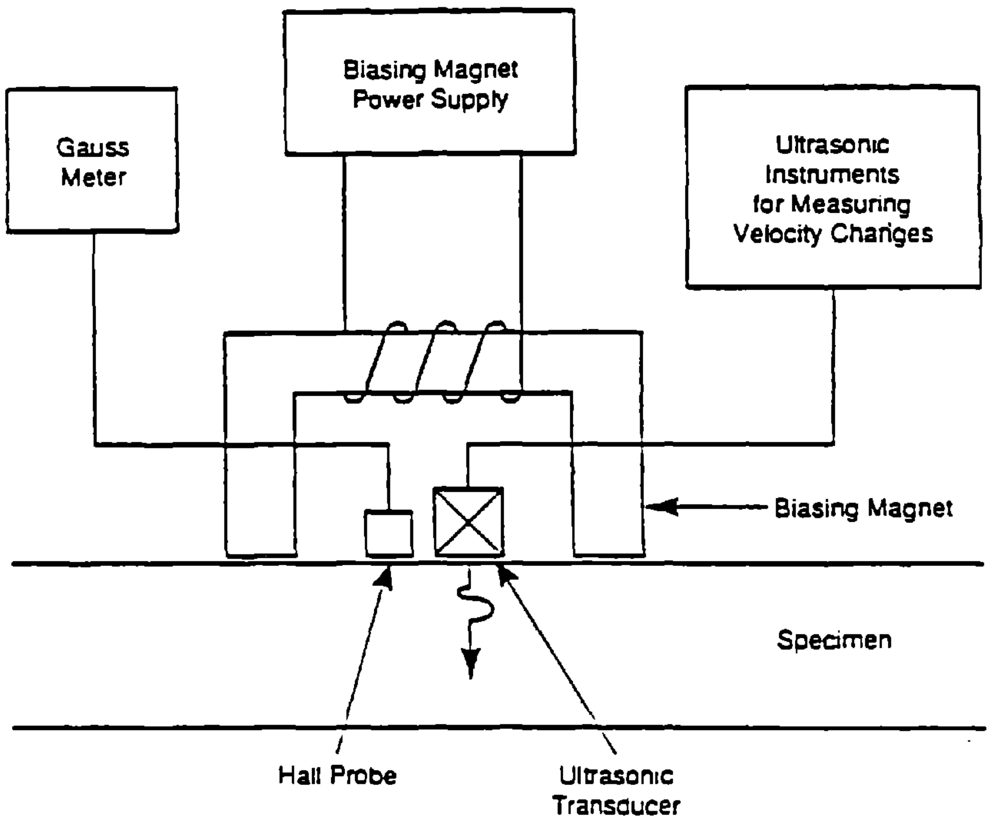


Figure 28. Instrumentation for Measuring Magnetically Induced Velocity Changes for an Ultrasonic Wave (28),(29),(30)

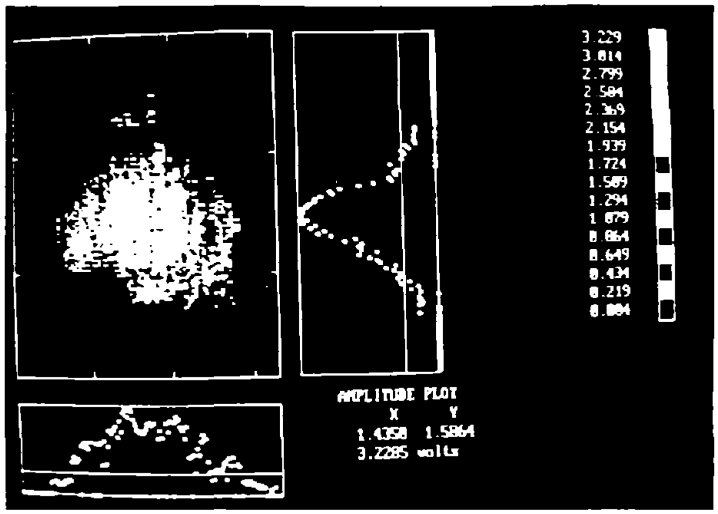


Figure 29. Amplitude Plot Before Processing (7)

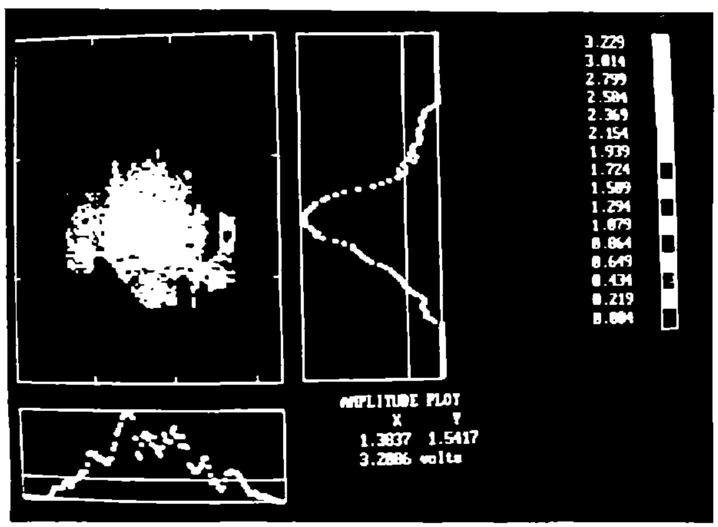


Figure 30. Amplitude Plot After Computer Enhancement (7)

STRAWMAN FOR NDE R&D TECHNOLOGY NEEDS

- Local Flaw Detection
  - Improvements in Conventional NDT Methods
  - Improvements in Advanced NDE Methods and Techniques
  - New, More Advanced NDE Methods and Techniques (e.g., for corrosion/corrosion rates)
- Global Monitoring
  - Improvements in Existing Methods (in particular AE)
  - Sensor Systems (e.g., vibration, remote viewing, and embedded sensors)
- Stress Measurement
- Signal Acquisition, Processing, Analysis, and Display

Figure 31. Strawman for NDE R&D Needs

ISSUES

- Codes, Specifications, and Standards
- Training
- QNDE -- Defect Characterization (Size, Shape, Orientation)
- Need for a "Systems Engineering" Approach
- Material Property Measurements (Fracture Toughness)
- Process Monitoring
- Performance Monitoring
- Improved NDT Methods and Techniques
- Sensors (Noncontact)
- Corrosion and Corrosion Rate Monitoring
- Technology Transfer
- Serviceability
- Life Prediction
- Progressive Damage
- Preventive/Predictive/Corrective Maintenance
- Predictive Models
- Model Programs (Aging Aircraft, Nuclear Power, Corps of Engineers)
- Specimens and Physical Standards
- Training/Testing Standards
- Reliability of Methods (User Friendly, Practical, Economical)
- FHWA Program Organization (Regional/State Centers)
- Seed Money for Basic Research on Structural Defects
- Record Keeping
- Information Dissemination
- Test Bed and Round-Robin Testing
- Input to Bridge Management Systems
- Guidelines for Sensors Installation
- Emphasis on Development (Follow-Up on Use, Especially for Field Worthiness)
- Design for Inspectability
- Accessibility of Areas Under Inspection
- Geometric Fingerprinting (Baseline and Follow-Up)
- Database Requirements (Data Format)
- Bridge NDE Information Database

Figure 32. R&D Issues for NDE of Steel Bridges

A. PRIORITIES FOR INSERVICE BRIDGES

1. MONITORING OF CRACK GROWTH
   - Development of Portable Monitoring Techniques, Such as Acoustic Emissions, Ultrasonics, Retrofit Fiber Optics Sensor Systems(???), VCR Monitoring, Shearography(???), Thermography, Electromagnetics, etc. -- Continuous vs. Periodic
   - Methods to Characterize Size, Shape, and Orientation
   - Stand-Alone Unattended Operation
   - Rapid Results (Within Days)
   - Inexpensive (Scaled to Applications)

2. MACHINE TO DETECT WELD TOE CRACKS UNDER PAINT
   - Portable, Compact, Weighs Under 10 Lbs., Visual and Audible Readouts, Durable
   - Crack Dimension on Order of 10 Percent Member Thickness
   - Permanent Record Ability
   - Permanently Mark Indications
   - Adaptable to Automation
   - Inexpensive
   - Candidate Technologies -- Eddy Current, Acoustic Emission, UT B-Scan, etc.

3. BMS -- GLOBAL MONITORING STRATEGY
   - Quantitative Measure of Condition
   - Risk Analysis of Consequences

4. SCOUR
   - Monitoring Capabilities During Flood Stage
   - Inspection Methods After Flooding
   - Study of Scour Holes and Different Sediment Effects

B. PRIORITIES FOR FABRICATION INSPECTION

1. AUTOMATED INSPECTION
   - Groove Welds
   - Fillet Welds
   - Partial Penetration Welds

2. QNDE
   - Look at Similar Programs in Related Agencies
   - Intuitively Obvious (to User) Systems

Figure 33. R&D Needs and Priorities for NDE in In-Service Bridges and Fabrication Inspection

CONCRETE BRIDGES: STATE-OF-THE-ART IN NDE TECHNIQUES

Allen G. Davis, Ph.D., D.Sc.
Principal Engineer
STS Consultants Ltd.
Northbrook, IL

1. INTRODUCTION

One of the earliest records of nondestructive testing of bridges using scientific methods celebrates its centenary this year; in 1892 a group of French Civil Engineers produced dynamic measurements of bridge deflections for different horse-drawn carriage loads. Not much progress was made in this field however until the 1940s, when the military applications of physical and chemical principles were extended to civilian life at the end of the Second World War.

Great enthusiasm was felt in the fifties and early sixties for the knowledge accruing about the physical and chemical properties of concrete from nondestructive tests in the laboratory, usually correlated with destructive tests on the same laboratory specimens. At the same time, equipment was being developed for the extension of NDT to concrete structures in the field, and many publications concerning the successful application of these test programs appeared.

I have to ask the question: ***why is it then, over thirty years later, that very few engineers today are familiar with NDT methods, and that the majority consider NDT to be of limited use when they have a problem to solve, let alone a routine inspection to make?***

2. CREDIBILITY IN NDT

Lack of credibility often comes from the engineer's personal experience, or from second-hand information about the experiences of others. Why do these problems in accepting NDT on concrete structures exist? The following list of possible reasons is not exhaustive, but in the author's experience form the basis for much of the skepticism.

a) Equipment Inadequacy

Most equipment until very recently was more suited to sterile, laboratory conditions, being cumbersome, unreliable, slow to operate, unsuitable for the accessing of difficult areas and producing data which had to be recorded manually for entry into a data base at a later time. Reproducibility of test data was poor, which resulted in a lack of statistical confidence in the results produced. The personal factor entered into testing, with skilled operators often producing different results than less able technicians.

b) Standardization and Codes of Practice

As a result of this, acceptance of NDT by Standard and Code setters such as ASTM and ACI was slow to materialize. Very few nondestructive methods have their own ASTM Standard at the present time, and those that do exist focus mainly on strength determination. This situation is changing rapidly, and NDE Committees are active in every code- and standard-setting body in the nation.

c) Subjectivity of Test Result Interpretation

Nondestructive testing by definition is indirect: the structure remains intact, and the Engineer can not necessarily confirm (either visually or by destructive means) the validity of the interpretation given by the NDT "expert". Most engineers are "doubting thomases" at heart, and require firm evidence before accepting a diagnosis. It is only at a later date, usually when repair contracts are underway, that the full extent of the success or otherwise of the NDT program can be evaluated.

d) Availability of Optimum Skills and Techniques

Claims made by commercial testing organizations for the ability of any one particular method to perform in a consistent and successful way are often exaggerated, and the degree of exaggeration usually increases with the decrease in the number of techniques available in the testing organization's armory. One example is the attempt to extend infrared thermography from the detection of corrosion delamination on the upper surface of bridge decks to the mapping of similar delaminations in bridge substructures such as columns and piers, often shaded from direct sunlight. Recent careful research (Ref.6) has shown that this technique used in this context can not succeed, because the thermal gradient within the structural element is never high enough to give valid results.

It is my contention that these problems still exist, and that despite rapid development in this field over the last ten years, the greatest brake applied to the successful application of NDT for the evaluation of concrete bridges is the reticence of engineers to accept the validity of a nondestructive approach to the problem. There is a need for education of the average engineer in the different methods and applications, supported by active participation of these engineers in demonstration projects organized by bodies such as the FHWA.

3. NDT TOOLS AVAILABLE TODAY

References 1, 2 and 3 describe the wide range of nondestructive tests available at the present time for concrete evaluation, and Table 1 presents a catalog of tests associated with the evaluation of different features of concrete bridge elements. Table 2 gives a list of the main Standards and Code of Practice for the more accepted nondestructive tests today.

An essential ingredient of all nondestructive concrete structural evaluation is visual inspection, which will not be dealt with here, except to say that advanced technologies such as imaging and expert systems should be used to enhance the information obtained from visual surveys.

This paper will also exclude global structure performance testing such as static and dynamic load testing, deflection monitoring and modal analysis (Ref.4). Global methods have not yet demonstrated the ability to detect significant flaws in structures in time for effective remedial action.

4. SAMPLING AND CORRELATION

For NDE surveys to be effective, the Engineer who ultimately has to rely on their interpretation must have fullest confidence in:

a) the chosen test method(s),
b) the test operator's skill and dedication,
c) representative test sampling,
d) good correlation of the NDT results with other sought-after parameters such as concrete strength and durability.

In addition, the NDE survey must offer certain cost and time savings over other investigative methods, or the Engineer will not be persuaded that there is any advantage in this approach.

It is very important that test programs are diligently cataloged, with careful records being made of survey details such as the test date and time, the weather conditions during testing, the operator's name, equipment serial numbers and calibration certificates. Modern data acquisition systems allow the entry of such data in computer form at the site. For this to work, the test position frequency and layout should be decided and mapped out on the structure before any testing begins. This helps in future data processing, particularly if test points are on a grid or linear pattern. Any on-going calibration procedures during testing must also be recorded.

The scope of work prior to testing must clearly define the sampling frequency required to give significant, meaningful results. This does not mean that some flexibility in testing is not possible; for instance, local areas where major swings in test values are observed should be the object of a more detailed examination at the end of the originally planned survey, if time and money allow.

Malhotra and Carino (Ref.1) give many useful guidelines to the number of samples required for any particular method to be significant. Refs.3 and 7 discuss the required number of tests when correlation of NDT results with concrete strengths obtained from cores is required.

In certain cases, the prescribed Standards do not help in recommending adequate sampling frequency. A classical example is the ASTM test method in C 876 on half-cell measurements for corrosion of reinforcing, where a test point spacing of 4 ft is proposed as suitable for most purposes. Subsequent to the publishing of this Standard, it has been observed by many workers that absolute values of half-cell potential do not yield an accurate assessment of the severity of the corrosion at the moment of testing, but rather that the potential gradient between test points is more informative (Ref.5). In this case, test point grid spacings should be no more than 2 ft, and very often, only 1 ft apart. In addition, this Standard does not refer to the value of potential gradient information, indicating a need for immediate updating.

5. IN-SERVICE PERFORMANCE AND NDT

In order to evaluate the deterioration of a structure over its life-time, the assumption has to be made that the as-built condition was perfect, unless evidence exists to the contrary. This highlights the need for base-line condition surveys using nondestructive methods at the end of construction (paying particular attention to reinforcement cover, concrete density and integrity), so that future in-service performance surveys can be correlated with a valid reference.

When setting up the condition survey, the distinction must be made between testing for strength and testing for durability or a combination of the two. In both cases, a minimum amount of destructive sampling will be obligatory to correlate the NDT findings with features such as strength, density and reinforcement corrosion. The locations for the destructive sampling should always be selected after the basic NDT program has been completed, and the samples should be chosen to give as wide a range of conditions as possible.

Problems arise when contract specifications for inspection testing are drawn up with very tightly defined testing methods and quantities, and with no flexibility for adjusting the program as information is gathered. This is particularly the case when the specialist NDT is subcontracted, and the controller on site is not the Engineer who developed the original specification. Many of these problems can be avoided if the inspecting organization has its own in-house testing capabilities, but this is often not justified for cost reasons. The testing specialists are often precluded from the preliminary visual inspections and studies which contribute towards the definition of the testing program, and alternative approaches are not easily considered at time of going to bid for the testing work.

6. NEEDS FOR RESEARCH AND IMPLEMENTATION

The physical and chemical principles defining the behavior of concrete in different environmental conditions are now fairly well established. A battery of nondestructive testing methods is available today, and researchers and equipment manufacturers are concentrating on improving testing equipment to be more easy to use (size, weight, testing speed, data storage) and to be more reliable. At the same time, Standards writers such as ASTM and ACI are producing procedural recommendations for many of these methods.

There is need however for unbiased research into the selection of optimal testing techniques and programs to address specific problems (as an example, bridge deck corrosion/delamination) with an accurate prediction of the probability of success before the start of testing. This includes:

a) an assessment of the success rate of the method(s) selected,

b) the degree of subjectivity, or operator influence in the interpretation of test results,

c) the amount of destructive sampling required for correlation, and

d) quantifying disruption to the bridge users during testing.

Research is also needed for the development of testing tools which can be reliably operated by trained technicians and engineers in organizations such as State Departments of Transportation, without operator-influenced results. Groups such as the FHWA could influence development of this approach by promoting research in this field, linked with training programs for operators when the equipment becomes freely available. This has been a long-term goal for a number of years, but not many nondestructive testing systems have reached this stage of development at the present time.

There is need for the development of automated and remote sensing devices, which can go where human operators find themselves in difficulty, or even occasionally in danger. One such device called the Lizard is under trial in France, where an automated unit carries various testing equipment such as pulse velocity, cover meter, and half-cell up vertical faces with difficult access.

Finally, if authoritative bodies such as the FHWA believe in the fundamental advantages of NDE, then the development of extensive training courses in the subject should be undertaken. Pressure should be placed by the Civil Engineering Industry on colleges and universities to introduce the next generation of engineers to the fundamental principles controlling nondestructive testing.

7. REFERENCES

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Table 1

BRIDGE ELEMENT	PROBLEMS	NDT METHODS
Foundations	Scour, Integrity	Parallel seismic, Sonic logging, Ground Radar
Abutments	Frost action, Alkali-Silica (ASR) Carbonation and Sulfates, Movement	Pulse velocity, Impact testing, SHRP ASR test, Surveying, Ground Radar
Piers	Frost action, ASR, Corrosion, Cracking Carbonation and Sulfates	Pulse velocity, Impact testing, SHRP ASR test, Sonic logging, Resistivity and Corrosion potential
Beams	Cracking, Corrosion Poor density and Lack of reinforcing cover, Deflection	Pulse velocity, Impact testing, Rebound hammer, Pachometer, Resistivity and Corrosion potential Dynamic load tests
Decks	Cracking, Corrosion and Delamination, Pavement wear	Pulse velocity, Impact testing, Chain drag, Ground Radar, Infra-red thermog.
Superstructure	Railing security, Corrosion, ASR	Corrosion potential SHRP ASR test
Special Case - Prestress	Cable corrosion, Lack of duct grout, Cracking	Ground Radar, X-Ray, Impact testing, Pulse velocity

TABLE 2

STANDARDS AND GUIDES FOR NDE OF CONCRETE

1. ASTM C 805 - 1985: The Rebound Number for Hardened Concrete (the rebound hammer).
2. ASTM C 803 - 1982: Penetration Resistance of Hardened Concrete (the Windsor Probe).
3. ASTM C 900 - 1987: The pull-out test (cast-in-place only).
4. ASTM C 215 - 1985: The resonant frequency of concrete samples.
5. ASTM C 597 - 1983: Standard Test Method for Pulse Velocity Through Concrete.
6. ASTM C 1050: The Break-off Test for Concrete.
7. BS 4408 - Part 1: Nondestructive methods of test for concrete - electromagnetic cover measuring devices. BSI, London.
8. ASTM C 876: Test Method for Half-Cell Potential of Uncoated Reinforcing Steel in Concrete.
9. ASTM D 4580 - 1986: Test Method for Measuring Delamination in Concrete Bridge Decks by Sounding.
10. ASTM D 4788: Impulse Radar.
11. SHRP -S/IR-90-001: Delamination Detection.
12. SHRP -C/FR-91-101: Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures.
13. BS 1881 - Part 205: Gamma-Radiography of reinforced concrete. BSI, London.
14. ACI Report 345.1R-92: Routine Maintenance of Concrete Bridges.

NONDESTRUCTIVE EVALUATION OF TIMBER

Robert J. Ross
Supervisory General Engineer
USDA Forest Products Laboratory[1]
Madison, WI 53705-2398

SUMMARY

Nondestructive evaluation of materials, the science of identifying the physical and mechanical properties of materials without altering their end-use capabilities, is valuable in defining relationships between properties and performance of materials. Nondestructive evaluation techniques using relationships have current industrial applications. They are used for machine stress rating of lumber and ultrasonic veneer grading for laminated veneer lumber manufacture. Studies using nondestructive evaluations have been used for in-place detection of decay in wood structures such as bridges, grandstands, piers and laminated arches. We have prepared a comprehensive report that reviews nondestructive testing of wood research and application techniques. This paper provides a brief synopsis of that report and other pertinent research that has been published since completing it.

1. INTRODUCTION

Nondestructive evaluation (NDE) of materials is, by definition, the science of identifying the physical and mechanical properties of a piece of material without altering its end-use capabilities. Such evaluations rely upon nondestructive testing (NDT) techniques or tools to provide accurate information pertaining to the properties and performance of the material in question.

[1]The Forest Products Laboratory is maintained in cooperation with the University of Wisconsin.

During the past 30 years, forest products researchers and the forest products industry have developed and used NDT tools for a wide range of applications -- from the grading of structural lumber to the in-place evaluation of the mechanical properties of individual members in wood structures. The USDA Forest Service, Forest Products Laboratory (FPL) recently published a report that reviews NDT techniques used with wood products (Ross and Pellerin 1991). The purpose of this paper is to provide a brief overview of that report and review research results published since its completion.

2. FUNDAMENTAL CONCEPTS AND PIONEERING RESEARCH EFFORTS

Nondestructive testing techniques for wood differ greatly from those for homogeneous, isotropic materials such as metals, glass, plastics, and ceramics. In such nonwood-based materials, whose mechanical properties known and tightly controlled by manufacturing processes, NDT techniques are used to detect the presence of discontinuities, voids, or inclusions. Because wood is a biological material, these irregularities occur naturally and further, may occur because of agencies of degradation in the environment. Hence, NDT techniques for wood are used to measure how natural and environmentally induced irregularities interact in a wood member to determine its mechanical properties.

Armed with this concept, early forest products researchers vigorously examined several techniques for grading structural lumber and evaluating the quality of laminated materials (Bell et al. 1950, Galiginaitis et al. 1954, James 1959, Jayne 1955, Jayne 1959, Hoyle 1961, McKean and Hoyle 1962, Senft et al. 1962). Out of these pioneering efforts evolved a hypothesis, founded on fundamental material properties, for establishing relationships between NDT parameters and static mechanical properties of wood products.

The fundamental hypothesis for NDT of wood materials was first presented by Jayne (1959). He proposed that the energy storage and dissipation properties of wood materials, which can be measured nondestructively by using a variety of static and dynamic techniques, are controlled by the same mechanisms that determine the mechanical behavior of such materials. As a consequence, useful mathematical relationships between these properties and elastic and strength behavior should be attainable through statistical regression analysis methods.

To elaborate on Jayne's hypothesis, consider how the microscopic structure of clear, straight-grained wood affects mechanical behavior and energy storage and dissipation properties. Clear wood is a composite material composed of many tube-like cells cemented together. At the microscopic level, energy storage properties are controlled by orientation of the cells and their structural composition, factors that contribute to elasticity and strength. Such properties are observed at frequency of oscillation in vibration or speed of sound transmission. Energy dissipation properties, conversely, are controlled by internal friction characteristics to which bonding behavior between constituents contributes significantly. Rate of decay of free vibration or acoustic wave attenuation measurements are frequently used to observe energy dissipation properties.

Early laboratory studies to verify Jayne's hypothesis were conducted with clear wood and lumber products. Jayne (1959) successfully demonstrated a relationship between energy storage and dissipation properties, measured using forced transverse vibration techniques, and the static bending properties of small, clear wood specimens. His technique was based on forcing a bending member to produce transverse oscillations. Pellerin (1965) verified the hypothesis using a free transverse vibration techniques and dimension lumber. Pellerin's effort differed from that of Jayne in that the specimens were allowed to oscillate in response to an initial deflection. No forcing function was applied. Kaiserlik and Pellerin (1977) furthered the hypothesis by using wave techniques (longitudinal oscillation) to evaluate the tensile strength of a small sample of clear lumber containing varying degrees of slope of grain.

3. CURRENT INDUSTRIAL APPLICATIONS

Machine stress rating (MSR) of lumber and ultrasonic veneer grading for laminated veneer lumber manufacture are two currently utilized industrial applications of NDT that developed from early research efforts.

Machine stress rating, as currently practiced in North America, couples visual sorting criteria with nondestructive measurements of the stiffness of a piece of lumber to assign it to an established grade (Galligan et al. 1977). Machines measure the modulus of elasticity (MOE) of individual specimens that are passed through endwise. They accomplish this by measuring the bending deflection resulting from a known load or by measuring the load required to accomplish a given amount of deflection. Special features of some machines include sensitivity to low-point elasticity, the ability to detect sections of lumber that may have MOE values much lower than the average of the piece. Design stresses are determined from machine MOE values, using a regression between MOE and strength.

Laminated veneer lumber manufacturing facilities use stress wave NDT techniques to sort incoming veneer into strength classes prior to processing the veneer into finished products. Veneers are assigned to strength categories, which are established through empirical relationships between stress wave velocity and strength, based on the velocity at which an ultrasonically induced stress wave travels through the wood (Sharp 1985).

4. CURRENT RESEARCH ACTIVITY

Considerate research activity has recently focused on applying NDT concepts to wood-based composites, decay detection, and in-place evaluation of wood structures. In addition, current research is aimed at enhancing current MSR techniques and developing low-cost MSR systems.

4.1 WOOD-BASED COMPOSITES

Successful verification of Jayne's (1959) hypothesis using stress wave techniques on wood-based composites has been shown by Pellerin and Morschauser (1974), Ross (1984), Ross and Pellerin (1988), and Vogt (1985). Pellerin and Morschauer (1974) showed that stress wave speed, a measure of energy storage properties, could be used to predict the flexural behavior of underlayment-grade particleboard. Ross (1984) and Ross Pellerin (1988) revealed that wave attenuation, a measure of energy dissipation properties, was sensitive to bonding characteristics and is a valuable NDT parameter that contributes significantly to predicting the tensile and flexural mechanical behavior of wood-based particle composites. Vogt (1985) furthered application of the hypothesis to wood-based fiber composites. In another study, Vogt (1986) also found a strong relationship between internal bond and stress wave parameters of particle and fiber composites.

4.2 DECAY DETECTION

Verification of Jayne's hypothesis with wood subjected to different levels of deterioration by decay fungi, which have a detrimental effect on the mechanical properties of wood and are commonly found in wood structures, has been limited to studies that have employed only energy storage parameters. Wang et al. (1980), for example, found that the frequency of oscillation of small, eastern pine, sapwood cantilever bending specimens was significantly affected by the presence of decay. Pellerin et al. (1985) showed that stress wave speed could be used successfully to monitor the degradation of small, clear wood specimens exposed to brown-rot fungi. They showed a strong correlation between stress wave speed and the compressive strength parallel to the grain exposed wood. Rutherford (1987) showed similar results. He also revealed that MOE perpendicular to the grain, measured using stress wave techniques, was significantly affected by degradation from brown-rot decay and could be used to detect incipient decay. Chudnoff et al. (1984) reported similar results from experiments that utilized an ultrasonic measurement system and several hardwood and softwood species. Patton-Mallory and DeGroot (1989) reported similar results from a fundamental study dealing with the application of acousto-ultrasonic techniques. Their results also showed that energy loss parameters may provide useful additional information pertaining to early strength loss from incipient brown-rot decay.

Acoustic emission techniques have also been investigated for use in decay detection. Utilizing a small sample of clear white fir specimens infected with brown-rot fungi, Beall and Wilcox (1986) showed a relationship between selected acoustic emission parameters and radial compressive strength.

4.3 IN-PLACE EVALUATION

Several organizations have published results of their efforts using these concepts for in-place evaluation of wood structures. Pellerin (1989) summarized his results and those of others (Browne and Kuchar 1985, Hoyle and Pellerin 1978, Hoyle and Rutherford 1987, Neal 1985) on successful use of stress wave methods for in-place detection of decay in wood structures. All studies used energy storage parameters from stress wave, time-of-flight-type measurement systems. Structures evaluated successfully included bridges, footbridges, grandstands for spectator activities, piers, laminated arches in school buildings, and the world's largest wood structure---TRESTLE.

Anthony and Bodig (1989) reported on the use of sonic spectral analysis techniques they developed and used for the inspection of wood structures. Murphy et al. (1987) and Dunlop (1983) reported on the use of acoustic and vibration techniques for evaluating wood poles. The technique developed by Murphy et al. (1987) involved measuring the vibrational response of a pole after it is tapped by a rubber mallet. Dunlop (1983) utilized an electronic system, sweeping through a selected range of excitation frequencies, to develop an acoustic signature of a pole. Resonant frequencies and their bandwidths were examined for use as NDT parameters.

The USDA Forest Products Laboratory (FPL) has worked on providing NDE techniques for two interesting problems. One involved developing a low cost technique to assess the residual strength of fire resistant treated roofing panels (Ross and others 1992). A series of techniques were investigated. The chosen technique consisted of a modified ASTM screw withdrawal test. The technique has been accepted by building inspectors and commercial equipment made available.

The FPL's other recent activity involves the development of a method to assess the structural integrity of wood members in the USS Constitution. Being the oldest commissioned ship in the US Navy, it is a significant part of US Naval history. Considerable effort is being devoted to using a variety of techniques to arrive at a useful NDE methodology (Witherall and others 1992).

4.4 MSR TECHNIQUES AND LOW-COST SYSTEMS

Research using noncontact scanning technology with conventional MSR systems is yielding encouraging results. This research has been aimed at providing more accurate estimates of the strength of lumber products and promises to significantly enhance the use of lumber in engineered applications (Bechtel and Allen 1987, Cramer and McDonald 1989).

Considerable effort is also being devoted to developing tools for low-cost MSR systems (Ross et al, 1991). This effort couples relatively inexpensive personal computer technologies and transverse vibration NDT techniques by innovative software programming. Tools developed from this effort promise to yield MSR systems at costs considerably lower than the costs of those currently used. This will make MSR lumber grading available to a wider range of wood products manufacturers and users.

Research has also been conducted to develop NDE methods for use in evaluating the quality of green or wet materials (Ross and Pellerin 1991, Ross and others 1992). This research has focused on determining if currently used NDE techniques can be used to locate wet wood, a special type of wood that results from aneotobic bacterial infection in living trees. It has also been aimed at predicting final product quality prior to drying.

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WHITE PAPER: COMPOSITE BRIDGES: NONDESTRUCTIVE EVALUATION OF ADVANCED COMPOSITE MATERIALS

Edmund G. Henneke, II

Engineering Science and Mechanics Department
Virginia Polytechnic Institute and State University
Blacksburg, VA 24061

ABSTRACT

The nondestructive evaluation of civil engineering structures such as bridges requires the synergism of at least two important components: (1) nondestructive test techniques that are capable of economically detecting the important flaws or damage and (2) physical-mathematical models that identify which flaws or damage are important and that relate the important attributes of the flaw or damage to the prediction of the remaining serviceability of the structure. Nondestructive evaluation of advanced composite materials currently lacks a well defined physical-mathematical model to meet the requirements of component (2). The NDT community has developed experimental techniques for detecting damage in composites without precise information as to what the important damage or flaw type is. This session reviews the present state of activity in the area of nondestructive evaluation of composites. The variety of nondestructive test techniques that have been applied to composites and the type of damage or flaws to which each NDT method is sensitive will be reviewed. Emphasis will be placed upon those techniques which might potentially be useful for scanning large areas in a reasonably short time. A final section will summarize where we presently stand in the detection of damage and flaws, where we need to go in the nondestructive evaluation of advanced composite materials, and what research needs to be performed to apply this knowledge to the NDE of composite bridges.

1. INTRODUCTION

Composite materials are constructed so as to take advantage of the most favorable engineering properties of each component of which the composite is constructed. As Jones [1] has noted, many properties can be improved by the formation of composite materials, including strength, stiffness, fatigue life, temperature-dependent behavior, corrosion resistance, thermal insulation, thermal conductivity, wear resistance, and weight. Obviously, not all of these properties can be improved simultaneously, and often, when one tries to improve one property of interest, it is at the expense of another. It is necessary to compromise when designing the material so as to optimize the end result.

Recently, composites have been identified as potential structural materials for use in bridges. In particular, composites have been identified for use in bridge decks and suspension cables, as well as for other structural components in bridge support structures. For application to already existing bridges, composite materials have been used for the repair of concrete columns and steel girders. Glass epoxy has been used to wrap around the top of concrete columns to reduce their tendency to "bloom." Graphite epoxy has been used to repair steel beams with fatigue cracks by adhesively bonding graphite epoxy plates over the known cracks. In the future, it has been suggested that non-critical components in present bridges of steel or concrete be replaced by composite parts to obtain experience with the use of this material. More exotic applications are often suggested for composites, such as the construction of long bridges over deep waters. It is likely that the use of composites in more ordinary bridges will prove to be of the greater benefit and broader usage.

While composite materials are more costly initially than the more common bridge materials (steel, concrete, and wood), they have many beneficial properties which should be considered in total life cycle costs. Composites have very good fatigue properties. When properly designed, composites have essentially infinite fatigue lives. Composite materials are corrosion resistant and can withstand road salts and other contaminants. They are very versatile and lend themselves to new, innovative manufacturing processes that could prove to be of benefit in the field for efficient construction of bridge structural components "on the scene." It is easy to visualize component modules being made from composite materials that would aid in rapid construction and repair of bridges.

The use of any new construction material is always approached conservatively until experience is gained with its long term performance. Thus, if and when bridges are first constructed from composites, it is likely that inspection will be performed at relatively frequent intervals to assure the bridge owner of the continuing safe performance of the bridge. However, it is evident from the many good properties of composites, that they will provide quality performance in the long term. Once designers and owners develop confidence in them, far less frequent inspection of in-service composite bridges will be required than is presently done for concrete, steel and wood bridges. Hence, in the future, the use of composite bridges could save long-term inspection costs and prove to have far lower life cycle costs than present bridges.

Composite materials such as glass epoxy or graphite epoxy are nonmagnetic. This is an important property for the construction of new, smart highways that will need bridges constructed of nonmagnetic materials for the proper operation of detection and navigating devices.

Increasing interest in the application of composites to engineering structures has led to an ever increasing awareness of the need to assure that the composite structure will perform safely at design conditions. More specifically, engineers have become aware of the need to nondestructively test and evaluate the mechanical condition of composite structures both after manufacture and during service to assure that the composite is capable of serving the design function.

Nondestructive evaluation of a material system requires the synergism of at least two important components: (1) nondestructive testing procedures which are capable of detecting and distinguishing all flaws or damage which are important to the mechanical serviceability of the structure, and (2) physical- mathematical models which are able to identify those flaws and damage that are important in predicting the remaining serviceability of the material/structure.

This paper reviews our knowledge of nondestructive testing of advanced composite materials. The NDT methods to be discussed include ultrasonics, acoustic emission, acousto-ultrasonics, radiography, eddy current, optical and thermal techniques. These techniques will be discussed according to the information that can be obtained about the damage state of a composite through their use, with emphasis placed upon those methods that might quickly scan large structures such as bridges. A common thread links these techniques. To learn as much as possible about the mechanical state of a composite, it is often necessary to use more than one of the available NDT methods. The use of multiple methods to obtain complementary information on the damage state is a well-recognized need in the nondestructive evaluation of composites. What one method may be incapable of detecting, another may find.

2. DAMAGE MODES IN COMPOSITES AND NDT

Generally, composite materials have many more damage modes than homogeneous metal and alloy systems. The most common damage modes have been identified as delamination, matrix cracking, matrix plasticization, fiber-matrix debonding, fiber breakage and component interface debonding. Each of these modes are commonly found in a composite material that has been subjected to environmental loading. The final catastrophic failure results from a complex interaction of the stress fields associated with each of these damage modes. This interaction will be different depending upon whether the composite structure is subjected to tension, compression, shear, torsion, or some combination of these loadings. The strength of the material at failure is also affected by the presence of porosity and voids resulting from inadequate manufacturing processes. Environmental conditions that can lead to damage include mechanical loadings (such as impact, fatigue, or static overloads), moisture absorption and ultraviolet degradation of the matrix material.

The presence and amount of damage in the composite at any time depends upon the history of the material. Presently, the major problem faced by the nondestructive evaluation of composite materials is the relation between detected damage and predicted mechanical serviceability. In addition to the need for in-service, continuous certification of composite bridges, nondestructive evaluation methods need to be developed for manufacturing process control of composites, initial certification of the completed bridge, and certification of repairs.

Many nondestructive testing methods have been developed that can detect most of the types of damage known to be present in composites, including delamination, matrix cracks and, to some degree, fiber/matrix debonding. A notable exception is the very important fiber breakage mode which obviously affects tensile strength. Once the damage has been detected, the major difficulty is evaluating the effect the damage might have upon the important mechanical properties of stiffness, strength or fatigue life of the material. It is not at all clear that what we detect and call "damage" is truly damage in the sense that it is detrimental to the serviceability of the material.

A simple test can be performed to illustrate the last statement. Consider a polymer matrix composite, e.g., a graphite epoxy laminate. Construct a specimen with dimensions of one inch wide by eight inches or so in length. A 1/4 inch diameter hole is drilled in the center of the specimen, and it is pulled to failure. The failure location will be at the narrowest cross section, i.e., across the width of the specimen at the location of the hole. The nominal tensile strength is defined by dividing the applied load at failure by the cross sectional area at the location of the hole. Now consider a second, "identical" specimen. Apply a tension-tension fatigue loading to the specimen of 70% of the previously determined nominal tensile strength for 100K cycles. After this loading, apply a quasi-static tensile load to failure and determine the nominal tensile strength as described previously. The nominal strength of the specimen that has been so fatigued can be as much as 10-20% higher than that of the unfatigued specimen, depending upon the laminate stacking sequence, the actual values of the maximum fatigue load and the number of cycles applied. It is easy to detect the presence of damage that develops around the hole in the fatigued specimen by several NDT techniques, including ultrasonics, x-ray radiography, or thermography. The damage serves to redistribute stress concentrations so the nominal strength has improved as a result of the "damage" that has developed around the hole. It should be noted, that if a third specimen is fatigued at high enough tensile loads, the damage around the hole can increase to the state where the specimen will fail upon the application of the next fatigue load cycle. That is, the damage grows to the point where the specimen can no longer sustain the applied fatigue load. The damage in the specimen can be beneficial up to a point and detrimental after some point. This is just a simple illustration of the difficulty found with detecting damage in composites and evaluating the effect of such damage upon the mechanical properties of the material.

On the other hand, the effect of certain types of damage or flaws in a composite is well known. Judd and Wright [2] investigated the effect of porosity in polymer matrix composites and concluded that the interlaminar shear strength of a composite decreases by about 7% for each 1% of voids up to a total void content of approximately 4%. Chatterjee and colleagues [3] assessed the significance of delamination defects and found these to be the most critical type of defect limiting the strength and lifetime of composites, especially under compression or flexural loadings.

The prediction of final catastrophic failure by a mechanics model has yet to be accomplished. It is evident from many typical X-ray radiographs that have appeared in the literature that damage developed as a result of fatigue loading is quite complex, consisting of numerous transverse cracks, longitudinal cracks, local delaminations, and, not evident in radiographs, fiber breaks [4]. While many investigators have been striving to develop a failure theory to relate the mechanical behavior to the damage condition of the material, at present no failure criterion analogous to fracture mechanics for homogeneous materials exists for composites. Thus the nondestructive testing community is faced not only with developing new test procedures for the testing of composite -- itself a difficult assignment because of the heterogeneity and anisotropic nature of the material -- but with the problem of not having a mechanics model that can be used to assess the damage found. Perhaps much of what is found and described as damage may not diminish the properties for which the material has been designed. How then do we evaluate the condition of the material?

From X-ray radiography, it is evident that damage states in a composite can consist of a large number of cracks and delaminations prior to final rupture. It is our contention that nondestructive test methods developed for the investigation of composites should be sensitive to the integrated effect of the total damage state in the material. That is, the total stress state existing in the material prior to failure is dependent upon the total state of damage. The stress concentrations existing in the neighborhood of crack tips are most certainly influenced by the stress concentrations developed by neighboring cracks and delaminations. Nondestructive test methods developed for composites should be more concerned with the total damage state than with the detection of individual cracks or individual appearances of other damage types, at least if the reason for performing the nondestructive test is to perform an evaluation and prediction of the remaining mechanical serviceability of the material.

One consequence of the need for nondestructive test techniques that are responsive more to the total damage state than to individual flaws is that one needs to change the standard approach to NDT. For example, a standard question asked of all NDT techniques when applied to homogeneous materials is "What is the resolution of the method? In particular, what is the smallest size crack that can be found?" For composites, this question is irrelevant. For composites, the question should be "What is the total state of damage?" Because of the different nature of the question when applied to composites, it is beneficial to categorize NDT methods for composites into those which provide detail on microstructural damage and those which provide macrostructural, or total field, information.

Conventional, well-established NDT methods can be, and are, used to inspect composites, but new methods for measuring and delineating nondestructive test parameters and data interpretation must be developed concurrently with an understanding of composite material behavior. Only in this manner can correct and optimum usage be made of the nondestructive testing information obtained.

3. NDT OF COMPOSITES

3.1 ULTRASONICS

Ultrasonics is the most widely used nondestructive test method for the examination of composite laminates. Depending upon the exact type of ultrasonic technique used, this method may be classified as either a microscopic technique or a macroscopic technique. For instance, if certain image enhancement procedures are used, relatively good detail can be obtained for delineating cracks in the matrix [5]. Also, the recent development of commercially available scanning acoustic microscopes provide the possibility of very high resolution microscopic images of the surface and near-subsurface of opaque materials [6-10]. On the other hand, other ultrasonic methods, such as standard C-scan, acousto-ultrasonics, attenuation, and velocity methods provide parameters which integrate over the entire field of damage. Ultrasonic methods are especially sensitive to porosity, matrix crazing and cracking, and delaminations. They are less likely to be sensitive to fiber breakage since the region of a broken fiber is many times smaller than the typical wave length used in ultrasonic testing. Ultrasonic methods have been used to measure velocities and elastic properties, to determine correlations between various ultrasonic parameters and strength, to detect the presence of damage before, during, and after loading, and to measure other properties such as moisture degradation, void detection, etc. A major difficulty with applying ultrasonic methods to inspect large structures such as bridges is that the technique is inherently slow and a coupling agent of some form must be used between the transducer and the specimen. Coupling agents can be eliminated with the use of laser techniques, but there are other difficulties involved with these techniques.

Composite materials are anisotropic. This fact must be taken into account in the interpretation of signals measured in ultrasonic tests. Several papers appearing in the literature [14-13] have measured the five elastic constants necessary to describe the orthotopic symmetry of unidirectional composite laminates. Some authors [14-17] have suggested using Lamb or plate waves to measure the effective, macroscopic elastic moduli of composite laminates. The advantage of this technique is that the moduli in the plane of the laminate can be determined by measuring the velocity of plate waves, if the correct analytical expressions are available. Since laminates are usually designed to carry loads in the plane of the laminate, determination of the in-plane elastic stiffnesses are of more importance to the mechanical serviceability of the laminate than are stiffnesses measured through-the-thickness by ultrasonic body waves. As an example of the use of Lamb waves in detecting damage, Tang and Henneke [17] showed that damage in composite laminates could be characterized by measuring the difference in the experimentally measured dispersion curves of the first order plate modes. This technique is quite time-consuming and unlikely to find an application in the inspection of large structures. However, such techniques might be used to investigate local areas that have been identified by more rapid methods.

Ultrasonic techniques have also been used to obtain correlation between signal parameters and material strength. Hayford, Henneke, and Stinchcomb [18] found a correlation between ultrasonic attenuation measurements and failure loads obtained from the standard short beam test for graphite polyamide unidirectional specimens. Vary, in a series of papers with co-workers [19-22], introduced a parameter he called the stress wave factor. This is a quantity which is related to the degree of attenuation of ultrasonic waves (the higher the stress wave factor, the lower the attenuation). Vary found that the stress wave factor was a sensitive indicator of tensile strength variations accompanying various fiber orientations and that this factor increased directly with interlaminar shear strength. This technique has been extensively studied in recent years and has come to be commonly called acousto-ultrasonics.

As Vary and Bowles pointed out [19], there was a need for nondestructive evaluation techniques that go further than simply finding overt flaws in composite structures. Even in the absence of such obvious flaws, the strength or endurance of the composite materials may be below the design or expected values. In perceiving the need for a new ultrasonic method for sensing and measuring variations in composite strength due to distributed material deficiencies such as microvoids and fiber-to-resin variations, Vary and Bowles looked for a method that would measure the relative efficiency with which stress waves would propagate in a given composite structure. Their working hypothesis was that the presence of microvoids, for example, would reduce both the composite strength and also the stress wave propagation efficiency.

The acousto-ultrasonics technique has some distinct advantages over other ultrasonic techniques. These include (i) Normal application of the method requires accessibility to only one side of the examined specimen. (ii) The transmitting and receiving transducer can be aligned parallel to the major load carrying direction. Thus, the measured ultrasonic parameters will be more directly responsive to the material condition in the direction of loading, as opposed to measuring through-the-thickness properties (iii). The acousto-ultrasonic technique provides a quantitative parameter that can be correlated to mechanical condition (of course, this is similar to ultrasonic attenuation measurements.)

Talreja, et al, [23] suggested transforming the received acousto-ultrasonic signal into the frequency domain and defining a set of shape factors calculated by forming various orders of moments of the amplitude-frequency curve. They showed a strong correlation between the plotted values of the zeroth moment and the change in stiffness that occurred in fatigue tests of a variety of graphite epoxy laminates. Close examination of the frequency spectra obtained from monitoring the acousto-ultrasonic signal at intermediate stages during the fatigue test showed that the higher frequency components contained in the signal at the start of the fatigue test were progressively attenuated. That is, the damage developed in the specimen as a result of fatigue interacted with the ultrasonic signal in a fashion to attenuate the higher frequency components. The greater the amount of damage, the greater the attenuation. This might be expected. The type of damage that develops in fatigue is transverse cracks, fiber/matrix disbonds, and fiber breaks. This damage, being at the microstructural level, would be expected to attenuate the higher frequency components more rapidly than the lower ones.

Other ultrasonic methods have been used for damage and flaw detection in composites. Ultrasonic C-scan techniques are used very often in composites to detect initial material flaws resulting during fabrication, damage resulting from static and fatigue loadings, and damage caused by foreign body impact. Generally, ultrasonic c-scans are unable to provide an image that is capable of providing information on the microstructural details of the damage in the specimen, and hence is classified as a macrostructural method.

Ultrasonic microscopy is a relatively recent arrival to the field of imaging small, microstructural differences in the surface, or near surface, of a material. Several different operating principles are presently in use, perhaps the most well-known of which is the reflection scanning acoustic microscope (SAM) developed by Quate and Lemons [6-9]. Acoustic imaging provides complementary information to that obtained from other imaging systems such as optical or electron microscopy. For example, Briggs [10] showed an example of an optical and an acoustic image of glass fiber reinforced epoxy material. Since both the matrix and the fiber are transparent, there was little contrast in the optical image. However, the variation in elastic properties of the two materials provided quite distinctive acoustic images.

Hollis, Hammer, and Al-Jaroudi [24] obtained very interesting images of glass fibers embedded in a polycarbonate matrix. In the latter case, since the velocity of longitudinal waves in the matrix material is only 2000 m/s, it was possible to image with high resolution below the surface, to a depth of 105 microns at 0.8 GHz. This is in contrast to the usual circumstance where high resolution scanning acoustic microscopy images only properties very near the surface of the specimen. Despite these successes of scanning acoustic microscopy, this technique is not applicable for the inspection of large structures such as bridges.

3.2 ACOUSTIC EMISSION

Acoustic emission has been used extensively to investigate damage development in composites since the number and intensity of acoustic emissions emanating from these materials under load is extensive (and usually audible.) The acoustic emission technique is categorized as a macrostructural technique. While each individual emission results from a separate event on the microstructural level, the signals received at the transducer are rapid, multiple, and affected by the total mechanical state of the material. While multiple transducers might, and are, used to locate the source of the emission event, the microstructural details of the damage in the material cannot presently be ascertained and detailed by monitoring the AE events.

As early as 1971, Mehan and Mullin [25] reported that each different failure mechanism --fiber fracture, matrix fracture, or debonding -- had different characteristic acoustic emission signal signatures. Speake and Curtis [26] concluded that the observed frequencies in the AE signals depended on the material type and geometry but that higher frequencies began to appear as fracture loads were approached. More recent work has indicated that the signal frequencies contain almost exclusively natural frequency components of the specimen-transducer system. Investigations are still on-going in attempts to make sense of the information contained in the AE signal characteristics. Reviews by Williams and Lee [27] and Duke and Henneke [28] have discussed a number of problems which must be solved before AE can be used in a routine fashion for the monitoring of the structural integrity of composites.

Historically, AE data were taken by using either count rate, total count, location detection, or spectrum analysis techniques. More recent work has utilized a variety of signal analyses. For example, several authors have reported success with the use of amplitude distributions to distinguish between different types of local failure modes [29,30]. These observations are still mainly empirical. Guild [31] has pointed out that no simple correlation can in general be expected as, for example, the amplitude of a fiber failure event depends upon the condition of the local fiber-resin interface, the extent of local debonding adjacent to the fiber fracture site, and a number of other possible factors.

The greatest success for the monitoring of structural integrity of composite materials has perhaps been found in applications to composite pressure vessels. In fact, a standard is currently employed for the proof testing of glass fiber composite tanks. The method is based upon the Felicity ratio which was introduced by Fowler [32,33]. The Felicity ratio is defined as the load at onset of AE activity divided by the maximum load previously applied to the specimen. The Felicity ratio may be less than, equal to, or greater than one, depending upon the material behavior. Fowler has developed an acceptance/rejection criterion using the Felicity ratio together with total counts, signal amplitude, and AE activity during a constant applied load.

Sundaresan and Henneke [34] suggested a proof test procedure for assessing the fatigue durability of a hip prothesis made of carbon fiber/thermoplastic matrix composite material. The suggested proof test involved a combination of fatigue cycling and static tensile loads applied to the structure. This proof test was found to provide an accurate indication of the fatigue life of the composite.

Acoustic emission techniques have potential for continuous, in-service monitoring of large structures such as bridges. Any gradual, or sudden, increase of AE activity with time could indicate the development of hazardous damage. It would be useful to consider the integral placement of optical fibers during manufacture to make the bridge a "smart structure." Fiber optics have been found useful in detecting acoustic emission in composites.

3.3 RADIOGRAPHIC METHODS

A variety of radiography techniques have been applied to the damage investigation of composite materials, especially neutron and x-ray radiography. Martin [35] used neutron radiography to measure the resin content, including the presence of porosity, resin-rich and resin-starved regions. Crane, Allinikov, and Chang [36], added boron fibers to the edges of graphite epoxy prepreg tape before specimen fabrication. They found that high energy x-ray radiography could then be used to examine ply orientation, laminate stacking sequence, fiber washing and waviness, ply overlaps, or ply underlaps. Predecki and Barrett [37] added crystalline filler particles to the matrix material and found that measurement of diffraction peaks could be used to determine residual strains and strains developed as a result of loading.

To apply x-ray radiography to investigate the microstructural details of damage in graphite fiber/polymer matrix composites, it is necessary to use low energy x-rays and/or to apply an x-ray opaque penetrant to the material. Both the polymer matrix and the graphite fibers are relatively transparent to x-rays and to obtain sufficient contrast in a radiograph to detect damage, a filler or penetrant that is more absorptive of radiation than the matrix or fiber must be used. Some researchers have been successful in showing that very low energy x-rays can be an effective means of detecting matrix cracks and delaminations (without use of a penetrant) if appropriate conditions are met. This technique has several advantages over the use of penetrant or filler techniques. First, it can locate damage that is not open to the surface. Second, nothing needs to be added to the material before fabrication. Third, it does not have the chemical effects (toxicity, degradation of matrix material, etc.) of most penetrants.

X-ray radiography is a technique capable of providing detailed information on the microscopic damage modes developed during loading. Chang and co-workers [38,39], for example, have used TBE enhanced x-ray radiography to find matrix cracks and delamination near stress concentration sites. Radiographs were made at various load levels statically and at various times during the cyclic history of fatigue loaded specimens. Chang reported that they were able to detect damage initiation, damage growth and type of failure mechanisms and that the radiographs could be used to study the actual stress redistribution by watching how the damage growth pattern changed. A recent review by Jones, et al, [40], discussed the application of several new radiography techniques to composites, including real-time video radioscopy, microfocus techniques, and single-sided X-ray backscatter imaging techniques. They point out that the single-sided backscatter techniques are particularly attractive because they require access to only one side of the examined structure and because the techniques offer excellent sensitivity to density variations.

For the investigation of the behavior of composites in the laboratory, low-energy radiographic inspection of polymer-based composites can reveal useful information about discontinuities such as voids, entrapped foreign materials, matrix cracks, resin-rich or resin-starved areas and other damage [40]. For large structures, however, radiography presents major practical limitations for economical application.

3.4 OPTICAL METHODS

Optical methods that have found application to the study of composite materials include visual, holographic and moire interferometry techniques. Depending upon the transparency or translucency of the material, simple visual observation can be used to detect matrix crazing, cracks and delaminations. Iversen, Schultz and Arnold [41] used thermal stressing, vibration stressing, and vacuum stressing to perturb the specimen surface so that holography could be used to find delaminations and cracks larger than 19 mm diameter in graphite epoxy. Other researchers have used thermal loads to perturb the specimen surface. Applications for bridge inspection would be found from diurnal solar heating. Erf, et al., [42] showed that holography could be used on composite aircraft structures outside the laboratory. Hence holography techniques have already proven their application to the inspection of large structures.

Oplinger, Parker and Chiang [43] found that the application of cemented film gratings for moire measurements provided an attractive approach for the exploration of the details of the mechanical behavior along the edges of a laminate. This is a particularly useful technique since many of the damage initiating mechanisms in composites occur at a free edge during loading. Moire interferometry has been used by Highsmith and Reifsnider [44] to study the characteristic damage state in graphite epoxy.

More recently speckle interferometry has been used to record very small translational displacements of the examined object. Shearography, a particular form of speckle shearing interferometry, has been used to measure strain induced by loading [45]. Shearography uses electronic image comparisons of a reference (unloaded) image with an image taken under load to provide indications of near surface flaws. The technique can inspect areas up to 3 x 4 feet in near real time.

3.5 THERMAL METHODS

During the past decade or so, a number of thermal methods have been developed and studied for the characterization of a variety of subsurface material properties in opaque solids, including composite materials. In particular, thermal wave imaging techniques, both real-time video and long-time scanning, have received a significant amount of attention. Real-time video techniques use commercial infrared electronic camera systems to scan and detect thermal patterns on the surface of objects with nonequilibrium thermal patterns developed by either the dissipation or conduction of heat through the object. Variations in the surface patterns are related to subsurface discontinuities or inhomogeneities. Also, commercial systems utilizing the thermoelastic effect have been developed to map the surface principal strain fields by monitoring the infrared emission synchronously with the maxima and minima of applied cyclic mechanical loads. Other techniques that create electronic images based upon the point by point detection of thermal effects have even more recently being studied and developed.

Thermal wave imaging basically uses a pulsed heat source to cause the development of pulsed thermal waves in a solid. The thermal waves in turn may be detected by a variety of acoustic or optical methods. If the heat source is rastered over the surface of the examined object, differences in the detected acoustic or optical signals can be used to "image" the conditions on, or just below, the surface of the examined object. A variety of methods using photoacoustic cells, photothermal methods, thermography and vibrothermography have appeared in the literature.

Real-time video thermography NDE methods are classified as macrostructural as they provide information about the integrated state of damage. Thermal wave imaging techniques, on the other hand, are capable of providing microstructural information on the material, at least in the vicinity of the surface.

Thermography is the general term given to the technique whereby contours of equal temperature--isotherms--are mapped over a surface. As a technique for nondestructive testing and evaluation, the application of thermography is based upon the assumption that defects, inhomogeneities, or other undesirable conditions of the test object will evidence themselves as local hot or cold spots in the isothermal mapping. To apply this technique one must, first, excite a thermal pattern in the test object to be studied; second, measure the areal temperature distribution on the surface of the examined test object; and, third, interpret the results according to well-established and understood physical principles [46]. Thermal methods for NDE and quality control has recently been reviewed by Henneke and Tang [47].

For the purposes of applying this technique, a thermal pattern can be excited in a solid material in a variety of ways [46-50]. Heat may be produced in an irreversible manner whereby thermal energy is generated by transformation processes which occur while the test object is being subjected to normal operating, testing or loading conditions. For example, an electrical component may overheat due to some abnormality when electrical current passes through it, or a structural material might develop hot spots under cyclic mechanical loading in regions where damage is occurring. For a true nondestructive test, low, non-damaging levels of mechanical vibrations can be applied to a material to excite the preferential development of heat around regions of discontinuities such as delaminations in composite materials. This technique has been named vibrothermography [51].

On the other hand, the test object may serve as a path for the conduction of heat. In this case, local inhomogeneities or flaws will cause local disturbances in thermal conductivities which will evidence themselves as gradients in the thermal heat patterns. This approach lends itself to the examination of structures exposed to solar radiation.

Of the various types of defects that can occur in composite laminates, in-service delaminations are probably the most dangerous because they can cause large reductions in compressive strength. Delamination can be produced by low energy impact by such events as stones thrown up from a runway or from dropped tools. While such impact can cause a significant amount of delamination, often only a slight indentation or no marks at all are visible on the surface where the impact occurred. This problem is of particular concern because damage is unlikely to be discovered unless the region is subjected to nondestructive inspection. A full scale nondestructive inspection is costly and time consuming.

A recent study has been shown that thermography is an effective method to inspect delamination in composite materials [52]. A large graphite epoxy filament-wound tank was sprayed by liquid nitrogen for a few seconds in a manner similar to spray painting so that the cooling of the surface would be uniform. Delaminations became visible on the infrared monitor as the surface cooled down. Good correlation was shown between the thermographic data and ultrasonic testing used to verify the results. In this case, this method has distinct advantages over other standard NDT techniques such as x-ray radiography and ultrasonics. X-ray radiography cannot detect subsurface delaminations in composites since there are no density variations between delaminated and undelaminated regions. In most instances, it is not possible to use an opaque penetrant as the subsurface delaminations have no open surface through which a penetrant might enter. Conventional ultrasonic inspection requires liquid couplant between the transducer and the material, which may degrade the composite, and requires time-consuming scanning techniques to cover the entire surface. Thermography can image a large area in the "real-time" scan of a television camera.

McLaughlin, et al, [48] induced thermal gradients in composite specimens by two heat generation techniques: conduction and direct heating methods. In the conduction technique, an edge section of the specimen was heated continuously by a heat source. Heat conducted into the test section caused transient thermal gradients in the plane of the specimen surface. In a direct heating method, a heat gun was used to apply heat directly onto the test section surface, causing transient thermal gradients perpendicular to the surface as well as parallel to the surface. These thermographic techniques were successful with graphite epoxy, glass epoxy and boron epoxy composites and aluminum in detecting flaws. The conduction test was able to detect delaminations, partial through-holes and surface cracks; the direct heating test was able to detect partial through-holes and delaminations.

Thermoelastic stress analysis is a very good technique for non-contact strain analysis when a cyclic load can be applied to the specimen. This technique is based upon the thermoelastic effect, which relates dynamic principal strain changes to the accompanied temperature changes. An advantage of thermoelastic measurement over other strain measurement devices, such as strain gages, is that it gives full field information. Large surface areas can be scanned relatively quickly to identify critical regions that can be later scanned more closely for better resolution. Also, high temperature environments pose little problem for this method. Limitations of the technique include: (1) it provides information only on the sum of the principal strains and (2) mechanical cyclic loads must be applied to the object.

Thermoelastic stress analysis has been used to detect and evaluate damage and defects in composites [53]. The full-field capability of this method is convenient for a relatively quick evaluation of the damage condition of a composite laminate. The thermoelastic stress analysis method is sensitive to manufacturing irregularities such as nonuniform resin distribution, void content, and ply thickness, and to service load damage such as matrix cracking and delamination. However, the surface ply dominates the thermal measurement [53]. Thus, damage in sub-surface plies may be concealed unless the effect of that damage is large enough to alter the thermoelastic emission of the surface ply.
Thermoelastic stress analysis has been used also to detect manufacturing irregularities in graphite epoxy composites [53]. A nonuniform fiber and matrix distribution results in irregularities in thermoelastic emission over the surface due to different material properties of fiber and matrix. Each constituent contributes to the local temperature change differently, depending upon the local volume fraction. In the same manner, a variation of void distribution also contributes to a nonuniform thermoelastic response.

The thermal wave imaging method has emerged also as an effective, non-contact method technique for the inspection and characterization of defects and structural features of a wide variety of materials. Thermal waves, by nature, are heavily damped, dying out in distances of the order of a wavelength or less. The heavily damped nature of thermal waves make this technique a prime candidate for the nondestructive evaluation of near subsurface defects in opaque materials. Practical difficulties presently limit the application of this method to the laboratory.

3.6 OTHER TECHNIQUES

Other techniques have been used to a lesser extent than those discussed in the foregoing for the nondestructive evaluation of composites. For example, eddy current methods, measurement of electrical resistance, measurement of dielectric constant and loss tangent, microwave absorption, etc. have been applied by various investigators. The measurement of stiffness of coupon specimens in the laboratory has been used as a means for real-time monitoring of damage development in composites. Stalnaker and Stinchcomb [54] were the first to suggest use of the edge replication technique to monitor development of damage along the edge of a coupon specimen, and it was as a result of these measurements that the concept of the characteristic damage state was developed [55]. Of these techniques, perhaps eddy current and microwave absorption methods have potential for economical scanning of large structures.

4. SUMMARY

The application of nondestructive testing to composite materials has progressed to the level where much of the damage developed as a result of processing or loading can be detected, especially if two or more different NDT methods are used together in a complementary fashion. A notable exception continues to be techniques capable of detecting fiber breakage in a quantifiable manner. Techniques such as ultrasonics, acousto-ultrasonics, acoustic emission, and thermography interact with the total damage state of the material and provide a macrostructural view of the mechanical condition. Quantitative parameters that are measured by these techniques have been shown in many cases to correlate well with important mechanical properties. Thus, it is likely that, with more complete understanding of composite behavior, such techniques will be the ones of choice for use in monitoring in-service performance. Techniques such as x-ray radiography, scanning acoustic microscopy and replication provide microstructural detail of the damage state. These techniques are important for laboratory use as they provide much information on damage initiation and progression and damage modes. Such techniques provide extensive information on the material and are very useful in aiding material developers.

At the Workshop sponsored by the Federal Highway Administration, August 25-27, 1992, several new problem areas were identified for research in nondestructive evaluation of composites for bridges:

> One of the noted advantages for the use of composite materials in bridges is the ability to manufacture composite bridge components on the site through a variety of manufacturing techniques such as, e.g., pultrusion. To assure quality and reduce costs of the finished product, it will be necessary to use NDE methods for continuous control of the manufacturing process. New techniques for embedded sensors (the manufacture of "smart" materials) could prove to be very useful in this area. Not only could sensors embedded during the manufacturing process be used to provide feedback control of the process, the embedded sensors could be used for continuous, in-service monitoring of the bridge.
>
> Composite components for bridges will by necessity require thicker wall sections than general composite parts presently used, for example, in the aerospace industry. These thicker parts will require study and modification of NDE methods presently developed and used for application to thin walled composites.
>
> Composite components for bridges will be generally three dimensional. An example of this is the three-dimensional composite structures used for bridge decking. These three-dimensional structures will require "invasive" NDT techniques such as currently used for inspecting piping in heat exchangers for the inspection to cover completely the interior of the component.
>
> New methods for repair of composite components, if and when necessary, will require development of NDE methods for certification of the repair.

It will be necessary to study scale-up of NDE methods presently used for laboratory sized specimens, or even structures as large as aircraft components to the size of bridge structures. Scale-up problems will also include problems with accessibility of the inspected component to the NDE sensor.

For the application of acoustic emission, which is very likely to be one of the most important techniques for monitoring composite bridges, new studies on signal analysis will be necessary. These will include the need for discriminating between valid AE signals from the bridge and extraneous signals from traffic or other noise sources.

With the new construction of bridges from composites, designers and NDE practitioners must be encouraged to practice "concurrent engineering." That is, they must be encouraged to work closely together at the very beginning of any design so as to assure inspectability of the final product. Since the construction of composite bridges is a new concept, it should be easier to do this from the onset. There should be no preconceived, fixed notions to be overcome. So often new ideas are difficult to place into action because of ingrained concepts of "it must be done this way, since this is the way we have always done it."

For application to the NDT of bridges, the techniques most likely to be found to be useful are those capable of rapid scanning of large areas. These techniques include acoustic emission, thermography, ultrasonics and optical methods such as holography or shearography. The ease with which embedded sensors, such as fiber optics, can be manufactured in composite materials, offers a major opportunity to study smart structures for composite bridges. Embedded sensors can be used to provide feedback for process control during the manufacturing stage and for continuous in-service monitoring. It will be necessary to study also the application of mobile "robot" NDT techniques where it is necessary to obtain local information about possible damage at critical locations in a bridge.

The major obstacle presently delaying the widespread, safe application of composite materials is the need for understanding the failure behavior of these materials. While much of the damage can be detected, it is by no means clear what effect, if any, such damage might have upon the serviceability of the composite. As has been pointed out, sometimes damage that develops as a result of loadings such as fatigue can improve the nominal strength of an engineering component by reducing stress concentrations around notches. There remains much to be done by investigators in the area of nondestructive testing: (i) continuing investigations to continue to improve our capability for damage detection; (ii) close association with mechanics investigators to aid in delineating the failure processes so that correct failure prediction models can be developed; (iii) continuing efforts in developing quantitative NDT methods which are responsive to the integrated state of damage and which produce test parameters that correlate with the net damage condition, and (iv) the development and application of new techniques, such as the mentioned thermal imaging methods, that can provide new and complementary information on the state of the material. For application of NDT to large civil engineering structures such as bridges, work must be performed to develop the NDT methods for economical, rapid scanning.

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LIST OF ATTENDEES

Conference on Nondestructive Evaluation of Bridges
Westpark Hotel - Rosslyn
August 25-27, 1992

Dr. M. Sherif Aggour
Civil Engineering Department
University of Maryland
College Park, MD 20742
301-405-1942

Dr. Iamd L. Algadi
Civil Engineering Department
200 Patton Hall
Virginia Tech University
Blacksburg, VA 24061
703-231-5262

Craig Ballinger
314 Ayito Road
Vienna, VA 22180
703-938-1057
FAX 703-938-1252

Bobby G. Batten
NASA/LaRC
Hampton, VA 23670
804-864-4992

Charles Bersch
Institute for Defense Analysis
1801 North Beauregard St.
Alexandria, VA 22311
703-578-2863
FAX 703-578-2877

Chun Sheng Car
4313 Rowalt Drive, #202
College Park, MD 20740
301-277-5215

Dr. Nicholas J. Carino
National Institute of
Standards and Technology
Building 226, Room B158
Gaithersburg, MD 20899
301-975-6063
FAX 301-869-6275

Lloyd R. Cayes
Federal Highway Administration-TFHRC
6300 Georgetown Pike
McLean, VA 22101
703-285-2463

Dr. Steven Chase
Federal Highway Administration
6300 Georgetown Pike
McLean, VA 22101
703-285-2442
FAX 703-285-2766

Chester Chen
Federal Highway Administration-TFHRC
6300 Georgetown Pike
McLean, VA 22101
703-285-2471

Ki Ho Cho
James Instruments Inc.
3727 North Keozie Ave.
Chicago, IL 60618
312-463-6565
FAX 312-463-0009

Dr. Ken Chong
National Science Foundation
Room 1108
1800 G Street, NW
Washington, DC 20550
202-357-9542
FAX 202-357-0167

A.V. Clark
National Institute of
Standards and Technology
325 South Broadway
Boulder, CO 80303
303-497-3159
303-497-5030

Dr. Gerardo G. Clemena
Virginia Transportation Research
Council
P.O. Box 3817
University Station
Charlottesville, VA 22903
804-293-1949
FAX 804-293-1990

Thomas Collins
Collins Engineers, Inc.
165 North Canal Street
Chicago, IL 60606
312-454-1060
FAX 312-454-1060

John Cuesta
Physical Acoustics Corp.
P.O. Box 1184
Herndon, VA 22070
703-758-0120
FAX 703-258-8692

Dr. Allen Davis
STS Consultants Ltd.
111 Pfingsten Road
Northbrook, IL 60062
708-272-6520

Shelia R. Duwadi
Federal Highway Administration-TFHRC
6300 Georgetown Pike
McLean, VA 22101
703-285-2472

Bruce R. Ellingwood
Department of Civil Engineering
Johns Hopkins University
Baltimore, MD 21218
410-516-8680
FAX 410-516-7473

Phillip Eng
New York State
Department of Transportation
47-40 21st St., 5th Floor
Long Island City, NY 11101
718-482-4706
FAX 718-482-4582

James Erler
SAIC 2-6-5
1710 Goodridge Drive
McLean, VA 22102
703-556-7247
FAX 703-893-6522

James Ernzen
Corps of Engineers
3909 Halls Ferry Road
Vicksburg, MS 31980
601-634-2382

Dr. Colleen Fitzpatrick
18198 Aztec Ct.
Fountain Valley, CA 92708
714-964-0324
FAX 714-964-9338

Jerry Foster
1381 Teaberry Lane
Severn, MD 21144
202-272-8676
FAX 202-272-8697

Dr. Karl Frank
6005 Ivy Hills Drive
Austin, TX 78759
512-471-4590
FAX 512-471-1944

Ian M. Friedland
Transportation Research Board
2101 Constitution Ave.
Washington, DC 20418
202-334-3238
FAX 202-334-2006

Dr. Mark A. Friesel
1852 Blue Heron Blvd.
West Richland, WA 99352
509-376-2235

Dr. Gongkang Fu
Engineering Research
Bldg. 7A-600, NYSDOT
Albany, NY 12232
518-457-5826
FAX 518-457-6513

Victor Gansham
1 Hunters Point Plaza
Long Island City, NY 11101
718-482-4708
FAX 718-482-4582

Dr. Al Ghorbanpoor
University of Wisconsin, Milwaukee
P.O. Box 784
Milwaukee, WI 53201
414-229-4962
FAX 414-229-6958

Herbert H. Gould
DOT/RSPA/Volpe National
Transpoatation Systems Center
55 Broadway
Cambridge, MA 02142
617-494-2134
FAX 617-494-3066

Terry D. Halkyard
Federal Highway Administration
400 7th Street, SW
Washington, DC 20590
202-366-6765
FAX 202-366-3938

Dr. Martin Hamstad
National Institute of
Standards and Technology,
Div. 853
325 Broadway
Boulder, CO 80303
303-497-7067
FAX 303-497-5030

Dr. George Hearn
University of Colorado
Boulder, CO 80309
303-492-6381
FAX 303-492-7317

Dr. Edmund Henneke
ESM Department
Virginia Polytechnic Institute
and State University
Blacksburg, VA 24061
703-231-3243
FAX 703-231-4574

Oscar Henriquez
200 Park Terrace Ct., #8
Vienna, VA 22180
703-242-1445

Dr. Sam Hersh
Sierra Matrix, Inc.
48890 Milmont Drive, #105D
Fremont, CA 94538
510-623-3690
FAX 510-770-8662

Dr. Conrad Hoeppner
Johns Hopkins University
102 Maryland Hall
Baltimore, MD 21218
410-693-7238

Dr. Kirk Huang
New York State Thruway Authority
200 Southern Blvd.
Albany, NY 12209
518-471-4221

Dr. Garrett R. Hyde
U.S. Bureau of Mines
810 7th St., NW, M.S. 6204
Washington, DC 20241
202-501-9281
FAX 202-501-9957

Dr. Rola Idriss
New Mexico State University
P.O. Box 3CE
Las Cruces, NM 88003
505-646-3818

Dr. Richard Johnson
1801 Maple Ave.
Evanston, IL 60201
708-491-4794
FAX 708-491-7105

William A. Johnson
High Energy Services
3012 Wavenley
Pala Alto, CA 94306
415-856-6966

Dr. Nicholas P. Jones
Department of Civil Engineering
Johns Hopkins University
Baltimore, MD 21218
410-516-7874
FAX 410-516-7473

John E. Judd
VIBRA Metrics Inc.
1014 Sherman Ave.
Hamden, CT 06514
203-288-6158
FAX 203-288-4937

Kyoung Sup Jung
12107 Sweet Clover Dr.
Silver Spring, MD 20904
301-572-7843

Gary Klein
Wiss Janney Elstner Assoc.
330 Pfingsten Road
Northbrook, IL 60062
708-272-7400
FAX 708-291-9599

Dr. Satish V. Kulkarni
Lawrence Livermore
National Laboratory
P.O. Box 808 (L-333)
Livermore, CA 94550
510-422-5856
FAX 510-422-3834

Susan Lane
Federal Highway Administration-TFHRC
6300 Georgetown Pike
McLean, VA 22101
703-285-2111

Dr. Michael Lauriente, P.E.
Goddard Space Flight Center
Code 410
Greenbelt, MD 20771
301-286-5690

Hanna Lu
Department of Civil Engineering
University of Maryland
College Park, MD 20742
301-405-1994

Bart W. Mancini
Federal Transit Administration
400 7th Street, SW
Washington, DC 20590
202-366-0192

Dr. George Matzkanin
4615 Trail Crest Circle
Austin, TX 78746
512-263-2106
FAX 512-263-3530

David Mazvrek
U.S. Coast Guard Academy
Department of Engineering
15 Mohegan Ave.
New London, CT 06320
203-444-8530
FAX 203-444-8546

Charles H. McGogney
Federal Highway Administration-TFHRC
HAR-10
6300 Georgetown Pike
McLean, VA 22101
703-285-2034
FAX 703-285-2379

Charles M. Minervino
A.G. Lichtenstein & Assoc. Inc.
17-10 Fair Lawn Ave.
Fair Lawn, NJ 07410
201-796-6550
FAX 201-796-5102

Vasant C. Mistry
Federal Highway Administration
400 7th Street, SW, HNG-32
Washington, DC 20590
202-366-4599
FAX 202-366-3713

James R. Mitchell, PAC
711 Cow Hill Road
Mystic, CT 06355
203-536-3380
FAX 203-536-3380

Dr. John C. Murphy
Johns Hopkins University
Applied Physics Labortory
Johns Hopkins Road
Laurel, MD 20723
301-953-6214
FAX 301-953-6904

Larry Olson
Olson Engineering, Inc.
14818 W. 6th Ave., Unit 5A
Golden, CO 80401
303-278-3232
FAX 303-278-3612

Roger Owen
SRC
3300 Keller Street
Santa Clara, CA 95054
408-980-9729

Dr. Roy F. Pellerin
Civil Engineering
Washington State University
Pullman, WA 99164
509-335-4591
FAX 509-335-7237

Christine Peterson
SRI International
1611 N. Kent St.
Arlington, VA 22209
703-247-8459
FAX 703-247-8569

Dr. Joe Plecknik
1880 San Anselini
Long Beach, CA 90815
310-598-0617

David W. Prine
918 N. 7th Ave.
Marywood, IL 62062
708-345-1428

Ron Purvis
2921 Telestar Ct.
Falls Church, VA 22042
703-698-9780

Dr. Henrique Reis
University of Illinois
Department of General Engineering
117 Transportation Building
1045 Mathews
Urbana, IL 61821
217-333-1298

Michael Ritter
1 Gifford Pinchot Dr.
Madison, WI 53705
608-231-9229
FAX 608-231-9592

Dr. Robert Ross
1 Gifford Pinchot Dr.
Madison, WI 53705
608-231-9221

Camille George Rubeiz
American Iron & Steel
1101 17th St., NW
Washington, DC 20036
202-452-7190
FAX 202-463-6573

Dr. John B. Scalzi
National Science Foundation
Washington, DC 20550
202-357-9542
FAX 202-357-0167

Dr. Charles F. Scheffey
1011 Westbriar Dr.
Vienna, VA 22180
703-938-0077
FAX 703-938-0077

Phillip Stolarski
5900 Folsom Blvd.
Sacramento, CA 95819
916-739-2307
FAX 916-739-4905

Paul Tan
10400 Ashcroft Way
Fairfax, VA 22032
202-272-8671

Dr. Cecil Teller
Southwest Research Institute
P.O. Drawer 28510
San Antonio, TX 78228
512-522-5961
FAX 512-684-4822

Dr. Graham Thomas
Lawrence Livermore
National Laboratory
P.O. Box 808 (L-333)
Livermore, CA 94550
510-422-7325
FAX 510-422-3834

Louis N. Triandafilou
31 Hopkins Pl., Room 1611
Baltimore, MD 21201
410-962-2460
FAX 41-962-3419

Ernesto Villalba
200 Park Terrace Ct., S.E., #8
Vienna, VA 22180
703-242-1445

Dr. Y. Paul Virmani
Federal Highway Administration-TFHRC
6300 Georgetown Pike
McLean, VA 22101
703-285-2439

John Warhus
Lawrence Livermore National
Laboratory
P.O. Box 808 (L-228)
Livermore, CA 94550
510-423-2783

Dr. Terry Wipf
Iowa State University
420 Town Engineering Building
Ames, IA 50011
515-294-6979
FAX 515-294-8216

Dr. Charles Woo
Federal Highway Administration-TFHRC
6300 Georgetown Pike
McLean, VA 22101
703-285-2444

William Wright
Federal Highway Administration-TFHRC
TFHRC
6300 Georgetown Pike
McLean, VA 22101
703-285-2496

Xiaoying Wu (Sherry)
Dept. of Civil Engineering
Room 1179, Bldg. 088
University of Maryland
College Park, MD 20742
301-405-1994

Dr. Ben T. Yen
Lehigh University, Fritz
Labortory
Departent of Civil Engineering
13 E. Packer Ave.
Bethleham, PA 18015
215-758-3536
FAX 215-758-4522

Robert C. Y. Young
Iffland Kavanagh Waterbury
1501 Broadway
New York, NY 10036
212-944-2000
FAX 212-302-4645