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Abstract

This report reviews quality assessment (QA) and quality control (QC) methods used during fabrication of fiber-reinforced bridge structures. In addition, potential candidates of nondestructive inspection (NDI) techniques that enable structural life-cycle performance monitoring are evaluated. The report establishes recommendations for QC/QA practices for FRP-based civil structures. Various bridge types and their substructures are introduced and critical defects are identified with respect to the type of structures and material used. Current and emerging technologies and their potential benefits and limitations are discussed. QA/QC and the various NDI techniques ensure that the product meets quality standards during construction and/or fabrication stages. The research will lead to a better understanding of FRP structures and thus to more composite candidates in the DelDOT system.

1. Introduction

Nondestructive inspection (NDI) techniques are needed to detect fabrication defects in the factory. Furthermore, our civil infrastructure deteriorates with time due to aging of the materials, excessive use, overloading, climatic conditions and lack of sufficient maintenance. Proper inspection methods are required to quantify these deteriorations in bridges that have traditional construction materials. Advanced fiber-reinforced polymer (FRP) materials represent a new construction material that does not corrode and requires less maintenance. However, bridge inspection is still required to ensure public safety. Inspection methods applicable to large-scale composites are needed for quality control and quality assurance (QC/QA) and field inspections.

The collapse in 1967 of the Silver Bridge in Point Pleasant, West Virginia, initiated the formal requirements for the inspection of highway bridges in the United States. More than 30 years later, visual inspection remains the primary method of bridge inspection, which is highly dependent on the skill and the dedication of the individual (Washer, 1998). The increased use of FRP in bridge application represents a challenge to the current inspectors who have minimal experience with these new materials and designs. However, many new and promising techniques for NDI have emerged in recent years, though it should be noted that only a few of them can be applied to civil engineering. Consistently applied, NDI is a reliable and objective method of determining bridge condition that can be used as a part of the Federal Highway Administration's Bridge Management Systems (BMS), which was implemented on federal system structures in 1996. BMS maintains data on the condition of bridge elements and processes the information through deterioration models so as to provide guidance on appropriating funds to maintenance, repair, rehabilitation, and replacement. Although the BMS framework exists, deterioration models do not exist for FRP bridge elements.

NDI, which is also known as nondestructive evaluation (NDE) or nondestructive testing (NDT), is extensively used in such industries as nuclear power and aircraft where high performance is needed and public safety is of paramount concern. The automotive industry employs NDI to improve quality and maintain competitiveness (Cartz, 1995). As the country's infrastructure ages, NDI applications are gaining popularity for repair and replacement decisions involving both safety and maintenance.

Recently, many reports and papers have been published concerning NDI techniques and their applications in civil engineering. Xanthakos (1996) gives a comprehensive examination of bridge strengthening and rehabilitation, including current NDI methods. Cordell (1998) suggests NDI to be a full-spectrum technology to provide in-process information as well as in-service characterization. Cordell also stresses the training or education required with NDI techniques. Extensive illustrations of principles and procedures of NDI are presented in a number of books (Bray, 1992; Blitz, 1991, 1996; Cartz, 1995). There also are books that address the topic of applications of NDI techniques to composite materials (Summerscales, 1987a, 1987b). Rens, et al. (1997, 1998) report that acoustic emission and ultrasonic tests are the most popular techniques among the state highway agencies based on a questionnaire sent to state highway organizations. Hearn, et al. (1998) suggest that NDI should be compatible with BMS to get more objective ratings or more reliable deterioration models.

Literature reviews about NDI cover bridge maintenance and rehabilitation (Blank, et al. 1996a, 1996b), emerging technologies of NDI (Thomas, et al. 1993), highway bridges (Washer 1998), steel bridges (Chase 1994), civil engineering (Bungey, 1994) and the NDI program of Minnesota's Department of Transportation (Beaudry, 1995). Martin (1998) addresses the accuracy of NDI techniques in bridge assessment in view of the wavelength of measuring signals. Gros (1998) introduces several NDI websites on the net.

This report reviews QA/QC, current inspection methods and advanced NDI techniques relevant to composite bridges. The QA/QC process includes design guides and the optimal manufacturing conditions, such as resin types, flow and cure control, and pretest or precaution for composite bridges.

This report is organized to review QA/QC, the current inspection methods and advanced NDI techniques relevant to composite bridges. QA/QC is based on design guidelines and provides optimal manufacturing conditions for resin types, and flow and cure control, as well as sets preconditions for composite bridges.

Various possible defects in composite structures, more specifically in FRP Bridge 1-351, are introduced and related to advanced NDI techniques. Current NDI technologies in civil engineering are investigated and promising NDI techniques and their potential benefits and limitations for composite bridges, including current NDI techniques at the University of Delaware's Center for Composite Materials (UD-CCM), are presented. The appendix of this report also reviews various types and components of traditional bridges, plus their potential defects and appropriate NDI techniques to detect such defects. The final appendix presents an example of the vibration NDI techniques that have been demonstrated on Bridge 1-351.

2. Quality Assurance/Quality Control during Bridge Fabrication

Prior to fabrication of a composite material bridge component, specification for materials, fabrication processes, and material testing techniques should be developed to ensure compliance with the engineering requirements. These specifications define the manufacturing process, materials, fabric lay-up sequence, and dimensions of individual components (e.g., face sheet thickness, foam core height). In order to achieve uniform and repeatable manufacturing of a product that meets design requirements, the incoming material is inspected. During processing, process parameters should be monitored and controlled to help ensure component quality. Following processing, nondestructive inspection of the bridge component can verify that the component meets certain design parameters. Destructive tests are limited to any witness plates that are processed at the same time. As part of the specification development, requirements for quality assurance of incoming materials, process monitoring and control, and post-processing inspection should be defined. Steps in the QA/QC process are shown in Figure 1. The parameters measured in each step are typical for a bridge component; however, critical parameters need to be considered in the development of specifications and inspection requirements in the design and development of each bridge component.

2.1 Inspection requirements – part of specification development

2.1.1 Critical parameters

As part of the specification development, parameters critical to the design and subsequent performance of the bridge component are identified. The specification includes tolerances on these critical parameters including dimensions and final structural performance. It should include limitations on variations in

- Geometry
    - Overall dimensions
    - Face sheet thickness
    - Core thickness

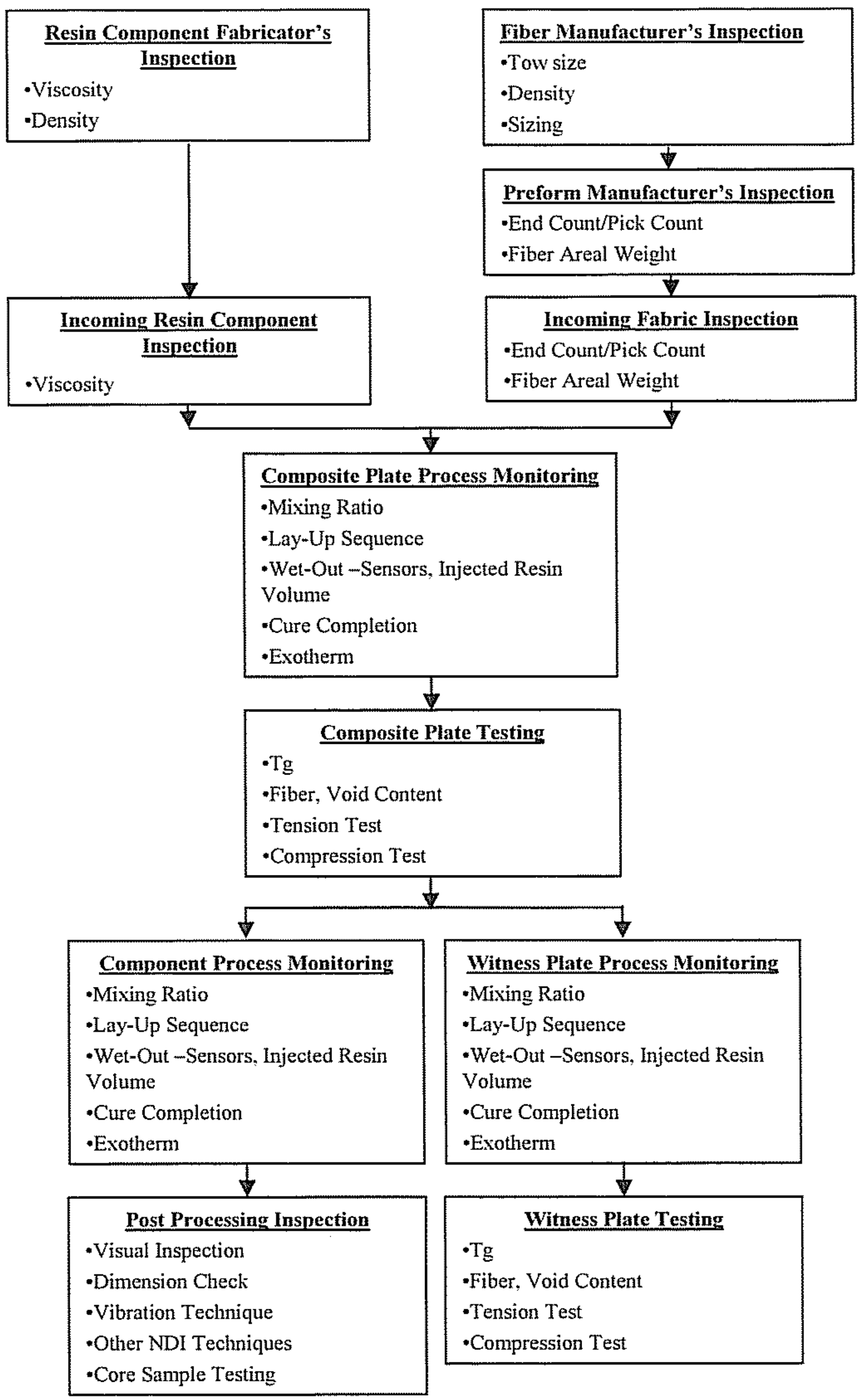


Figure 1 summarizes the QA/QC plan during manufacturing:

- Material property requirements
    - Face sheet stiffness
    - Glass transition temperature (Tg)
    - Fiber volume
    - Void fraction
    - Tensile strength and modulus
    - Compressive strength

These requirements were incorporated into the design including the selection of material and the preform stacking sequence. The function of QA/QC is to ensure that these requirements are met by each fabricated bridge component.

2.1.2 Statistical limits

Statistical limits are used as part of the decision process for each step of QA/QC. Acceptance testing of incoming materials uses statistical limits to ensure that the material used to fabricate the bridge component is the same as the material used in developing the design data. Statistical limits are also used to evaluate the fabricated bridge component and witness plates fabricated at the same time.

The establishment of statistical limits on critical parameters depends on testing during the design phase. The criteria used to determine equivalency between the design database of a given composite material and a subsequent test sample of the same material are selected based on the material properties of interest. For strength properties, the appropriate critera must reject either a low mean or a low minimum individual value. The appropriate statistical method for strength properties is given below as "Test for Decrease in Mean or Minimum Individual." This test was developed to have equal probability of rejecting a "good" set of data with either the test on the mean or on the minimum individual property. This balance between the two conditions of the test gives the maximum "statistical power," and is an improvement over the ad hoc methods used in industry to set material specification acceptance limits.

For modulus or some physical properties such as per ply thickness, the appropriate criteria require that the mean value be within an acceptable range; neither a high nor a low mean is desirable. The criteria for these properties are designed to reject either a high or a low mean value. The appropriate statistical method is given below as "Test for Change in Mean."

For certain chemical and physical properties, such as volatile content or porosity level, the appropriate criterion must reject a high mean value, as the desired property value is 0. The appropriate statistical method for these properties is given below as "Test for a High Mean."

Test for Decrease in Mean or Minimum Individual – The mean and standard deviation are approximated by x and s, from the individual test condition (environment) of the original material qualification database. The mean value from the incoming lot of material must meet or exceed

$$w_{mean} = \bar{x} - k_n^{mean} s$$

The minimum individual value from an incoming lot of material must meet or exceed

$$w_{\min} = \bar{x} - k_n^{\min} s$$

The mean and minimum individual k_n values are found in (Tomblin et al, 2001; Vangel, 2001). Simulations have been conducted using the combined mean and minimum individual value tests (Vangel, 2000). Simulated data for good material are shown in Figure 2. Note that even for good material, occasionally incoming lots will not pass the acceptance criteria. Retest criteria are appropriate for these cases. Simulated data for bad material (where the standard deviation for the incoming material population has increased by 50%) are shown in Figure 3.

The mechanical and chemical properties of material specimens are subject to random variability. Hence, one must accept the possibility of making an error in declaring a "good" material property to have failed a statistical test. For a fixed number of test specimens in a sample, the probability of this undesirable event occurring (defined as α in the following statistical tests) can only be made small at the expense of decreased likelihood of detecting failures in material when failure should be declared. The selection of the value for the probability of failing a statistical test in error, α, is a compromise between the two types of errors. If the statistical tests are being used with test programs where retests of "failed" properties are allowed, then a slightly higher value for α can be used, as the α after one retest will be effectively 2α.

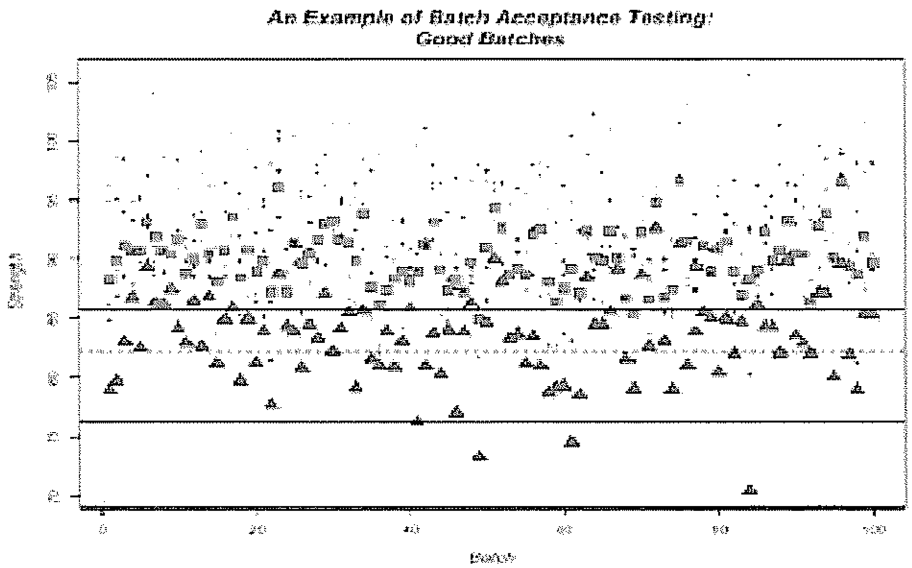


Figure 2 Example of Batch Acceptance Testing. The population of the incoming material has the same mean and standard deviation as the design data. The mean of each incoming lot is shown as a red square and the minimum value for each lot is a blue triangle. The upper solid line is the limit on lot mean and the lower solid line is the limit on lowest datum per lot. The dashed line is the B-value.

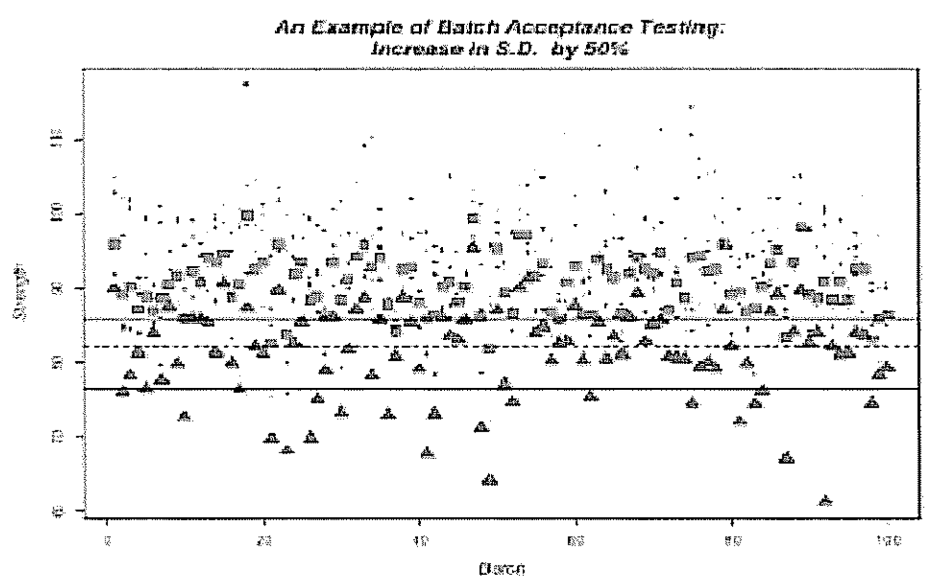


Figure 3 Example of Batch Acceptance Testing. The population of the incoming material has the same mean as the design data but the standard deviation has been increased by 50%. The mean of each incoming lot is shown as a red square and the minimum value for each lot is a blue triangle. The upper solid line is the limit

on lot mean and the lower solid line is the limit on lowest datum per lot. The dashed line is the B-value.

Test for Change in Mean - Since the sample sizes of the design database, n_d, and the new data sample, n_i, are different, a pooled standard deviation, S_p, is used as an estimator of common population standard deviation.

$$s_p = \sqrt{\frac{(n_d - 1)s_d^2 + (n_i - 1)s_i^2}{n_d + n_1 + 2}}$$

Using the pooled standard deviation and the mean values of the original and new datasets, the test statistic, t_0, is calculated using:

$$t_0 = \frac{\overline{x}_d - \overline{x}_i}{s_p\sqrt{\frac{1}{n_d} + \frac{1}{n_i}}}$$

Since this is a two-sided t-test, the required t value is $t_{a,n} = t_{\alpha/2, n_d - n_i - 2}$. Note that $a = \alpha/2$ for the two–sided test and $t_{a,n}$ is obtained from (Vangel, 2001).

For a material to pass the this test, the test statistic, t_0, must satisfy

$$-t_{\alpha/2, n_d - n_i - 2} \le t_0 \le t_{\alpha/2, n_d - n_i - 2}$$

Test for a High Mean - This test is designed to detect undesirably high mean values such as in the case of volatile content of prepreg. The mean of the incoming material property is said to be less than or equal to the mean of the design property if

$$t_0 \le t_{\alpha, n_d - n_i - 2}$$

where

$$t_0 = \frac{\overline{x}_d - \overline{x}_i}{s_p\sqrt{\frac{1}{n_d} + \frac{1}{n_i}}}$$

This is a one-sided t-test so $t_{a,n} = t_{\alpha, n_d - n_i - 2}$. Note that a=α for a one-sided test and $t_{a,n}$ is obtained from (Vangel, 2001).

Recommended Values for α - For determining batch acceptance limits for material specifications, it is recommended to set the probability of rejecting a good property (α) to 0.01 (1%) for all test methods that utilize the test statistics. A minimum of five specimens for strength properties and three specimens for modulus properties is recommended for material batch acceptance testing.

2.1.3 Quality control on incoming materials

The main material components of a FRP bridge are resin, fabric, and, frequently, core material. The processing specification may require additional materials, e.g., peel ply, distribution material, and breather cloth for VARTM manufacturing; these materials are not integral parts of the structural component. Variation in either component or process material can change the quality achieved during manufacturing and thus must be minimized. All incoming material should be checked for quality. MIL-HDBK-17, Volume 3, Chapter 3 (MIL-HDBK-17, 1997) provides guidance for quality assurance, primarily directed to aircraft applications using preimpregnated material (prepreg). This resource has been used as a starting point for other applications and material forms.

During the design process, specimens of the candidate material should be fabricated and tested to determine mean and coefficient of variation of critical material properties. These measurements determine the material allowables. The material specifications are based on *protecting* the material allowables, ensuring that the properties of the incoming materials will produce a composite material with properties that are not degraded significantly from the materials allowables. Acceptance criteria must be specified to assure that production parts will be fabricated with materials that have properties equivalent to the materials used to develop the allowables.

The user material specifications should require the suppliers to provide data that each production lot of constituent material meets the specification requirements for that material. The tolerances on the specifications for the constituent materials should be sufficiently tight so that the composite material allowables are protected. Overly stringent specification tolerances can affect price without providing improvement in performance. Acceptance test requirements may vary from user to user. However, the tests must be sufficient to assure the material will meet or exceed the engineering requirements. Acceptance tests for glass/vinyl ester are proposed in Table 1, a modification of Table 3.2.1 (MIL-HDBK-17, 1997).

Note that Table 1 is divided into two parts. The first part concerns uncured constituent. The purpose of these tests is to assure that the resin, fiber, and preform are within acceptable limits.

The second part involves tests on cured material. Tests on composite plates prior to processing of the bridge component help assure the properties of the bridge will meet design values based on the actual resin, fiber, and preform lots that will be used on the bridge.

TABLE 1 *Typical acceptance tests required for suppliers and users, modified from (MIL-HDBK-17, 1997).*

PROPERTY	TESTING REQUIRED		SPECIMENS REQUIRED PER SAMPLE
	PRODUCTION ACCEPTANCE (SUPPLIER)	PRODUCTION ACCEPTANCE (USER)	
Resin Component Properties			
Viscosity	X	X	-
Density	X		3
Fiber Properties			
Tow size	X		-
Density	X		3
Sizing	X		3
Tensile strength	X		3
Preform Properties			
End Count	X	X	-
Pick Count	X	X	3
Fiber Areal Weight	X	X	3
Composite Material Properties			
Fiber Volume		X	3
Void Content		X	3
Per Ply Thickness		X	1
Glass Transition Temp		X	3
Tension Strength & Modulus		X	6
Compression Strength		X	6
SBS or V-Notch Shear		X	6

The mechanical property tests are selected to reflect important design properties. The tension test evaluates the fiber strength and modulus. The compression test evaluates the reinforced fiber/resin combination. A shear test as a resin evaluation is optional, depending on the end product's emphasis on interlaminar or in-plane properties.

Note that a significant decision during the development of the QA/QC procedures is whether to fabricate and test composite plates prior to component fabrication, or to fabricate and test witness plates fabricated concurrently with the components, or both.

Factors to consider are the relative costs of test plate fabrication and testing as compared to component fabrication, number of components and their fabrication schedule, and the organization's experience and confidence in composite materials. The use of test plates fabricated before or concurrently with the components is somewhat difficult to judge since the thickness of the test plate is generally different from the component. A number of effects are driven by the difference in thickness. The thermal cycle during curing will be different. It may be possible to control the differences but for some processes, such control is very difficult. The preform lay-up may well differ. The stress state in the test panel may not be typical of the stress state throughout the component. The differences between the test plates and the components, as well as the use of the test plate data, should be considered as factors in establishing the QA/QC plan.

2.1.4 Monitoring techniques during processing

The bridge component fabricator generally has the responsibility for verifying that the fabrication processes are carried out according to engineering process specification requirements. This encompasses a wide range of activities described below to control the fabrication process. This section is an adaptation for bridge components of MIL-HDBK-17, Vol. 3, Section 3.2.2 (MIL-HDBK-17, 1997), based on aircraft components.

Material Control: As part of control of the fabrication process, the material must be under control.

Materials are properly identified by name and specification.

1. Materials are stored and packaged to preclude damage and contamination.
2. Perishable materials are within the allowable storage life at the time of release from storage and the allowed work life at time of cure.

Materials Storage and Handling: The user material and process specifications set procedures and requirements for storage of raw materials to maintain acceptable material quality. It may be necessary to store some of these materials at low temperatures, usually 0°F or below, in order to retard the reaction of the resin materials and extend their useful life. The supplier and user should discuss and agree on storage requirements and the shelf life of all materials.

Tooling: The tooling (molds) to be used for lay-up is subject to tool proofing/qualification procedures. This demonstrates that the tooling is capable of producing parts that conform to drawing and specification requirements when used with the specified materials and lay-up and processing methods. Tool surfaces must be inspected before each use to ensure that the tool surface is clean and free of conditions that could contaminate or damage a part.

Facilities and Equipment: The user will establish requirements to control the composite work area environment. These requirements are a part of the user's process specifications. The requirements should be commensurate with the susceptibility of materials to contamination by the shop environment.

Contamination restrictions in environmentally controlled areas typically prohibit the use of uncontrolled sprays (e.g., silicon contamination), exposure to dust, handling contamination, fumes, oily vapors, and the presence of other particulate or chemical matter which may affect the manufacturing process. Conditions under which operators may handle materials should also be defined.

In-Process Control: During lay-up of composite parts, certain critical steps or operations must be closely controlled. Requirements and limits for these critical items are stated in the user process specifications. Some of the steps and operations to be controlled are listed below:

1. Verification that any release agent has been applied and cured on a clean tool surface.
2. Verification that perishable materials incorporated into the part comply with the applicable material specifications. The resin system may consist of two or more components. The shelf life for all components has to be verified before mixing of the reacting systems occurs. The viscosity of each of the individual components can be measured and the mixing ratio should be documented.
3. Inspection of lay-ups to assure that engineering drawing requirements for number of plies and orientation are met.
4. Inspection of core installation, if applicable, and verification that positioning meets the engineering drawing requirements.
5. The user paperwork should contain the following information.

a. Material supplier, date of manufacturer, lot number, and total accumulated hours of working life.
b. Processing parameters including pressures, part temperatures, and times.
c. Part identification.

Part Cure: Requirements must be defined in user process specifications for processing parameters. These parameters depend on the process used to fabricate the part. For VARTM, parameters include heat rise rates, times at temperature, cool-down rates, temperature and pressure tolerances, and temperature uniformity surveys in the autoclave or ovens.

Witness Plates: Witness plates are test panels laid up and cured along with bridge components. After cure, the witness plates are for physical and mechanical properties to verify the parts they represent meet the engineering properties.

During infusion flow sensors will ensure complete wet-out of the preform. Flow sensors can be embedded in the preform at critical points (corners, locations where resin is expected to flow last). Tool-mounted sensors would allow for repeatable monitoring of the flow behavior and comparison between various impregnations. Examples of flow monitoring equipment includes

- Ionic conductivity based systems
    - Tool-mounted and embedded versions
    - Inexpensive
    - Large number of point sensors possible (>1000 sensors)
    - Industrial accepted
    - Drops dramatically during onset of cure; thus, cannot measure complete cycle.
- Dielectric principle based systems
    - Tool-mounted and embedded versions
    - Expensive
    - Medium number of point sensors possible
    - Measures through gelation to final cure; capable of measuring complete cycle

Other sensors include time-domain reflectometry and fiber optics.

The injected resin volume has to be monitored and has to be consistent with the preform setup.

Cure sensors similar to the flow sensors can be embedded or tool-mounted. They monitor the gelation and cure behavior of the composite and ensure a minimum degree of conversion of the resin system. Some of the flow can be used for dual purposes as cure and flow sensors. Examples of cure sensors include

- Ionic conductivity based systems
- Dielectric principle based systems
- Fiber optics
- Ultrasonic based systems
- Face sheet thickness sensor

2.1.5 Post-processing inspection of bridge component

After cure, the final dimensions of the part have to be checked. The face sheet thickness can be evaluated at various locations with ultrasonic thickness gauge. Transportation should be avoided until the part is completely cured. Tests on small samples ensure final void content (density measurements, photo micrographs), mechanical properties (fatigue, tensile, etc.) and complete cure (DSC, DMA). Visual inspection to detect dry spots and thickness variations out of specifications should be followed by a vibration NDI as outlined in the next chapter. Further evaluation with other techniques can become necessary when defects are found with these techniques.

Vibration NDI is recommended for identifying locations with potential damage (See Appendix C, Section 7). When locations where damage is a possibility have been identified, other forms of NDI can be used to further evaluate these areas.

2.1.6 Testing for critical parameters on witness plates

Witness plates made at the same time that the bridge components are fabricated reflect the ambient conditions and the resin mix of the bridge component fabrication. The plates will be of a thickness appropriate for standard test methods. Consequently, temperatures in the witness plates during processing will not be the same as the thicker bridge component. However, the witness plates do serve to verify that the materials were implemented according to the design plan.

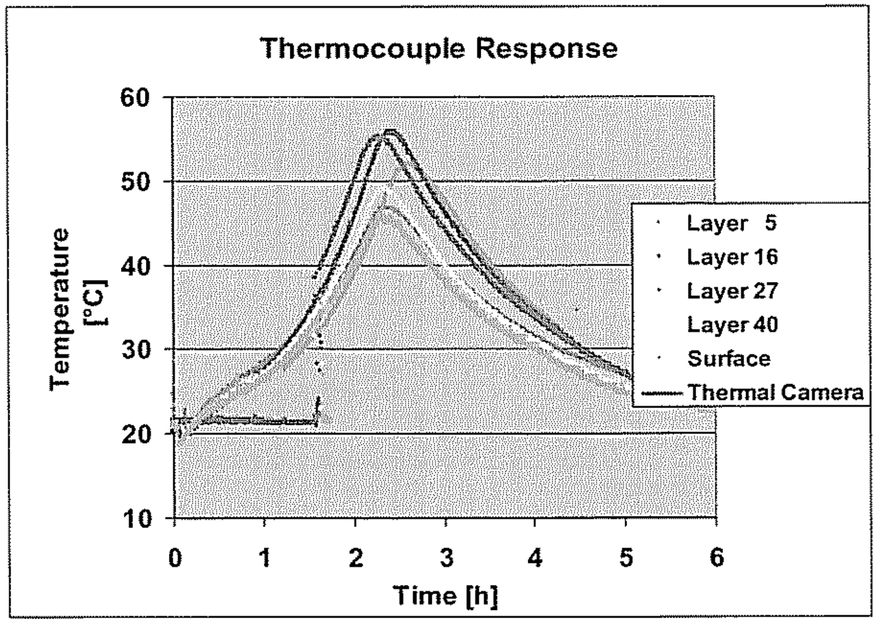
2.2 Implementation of process monitoring

Process monitoring should be implemented according to the requirements established in the materials and processing specifications. Additional considerations involved in the implementation include embedding of thermocouples to evaluate the exothermal potential.

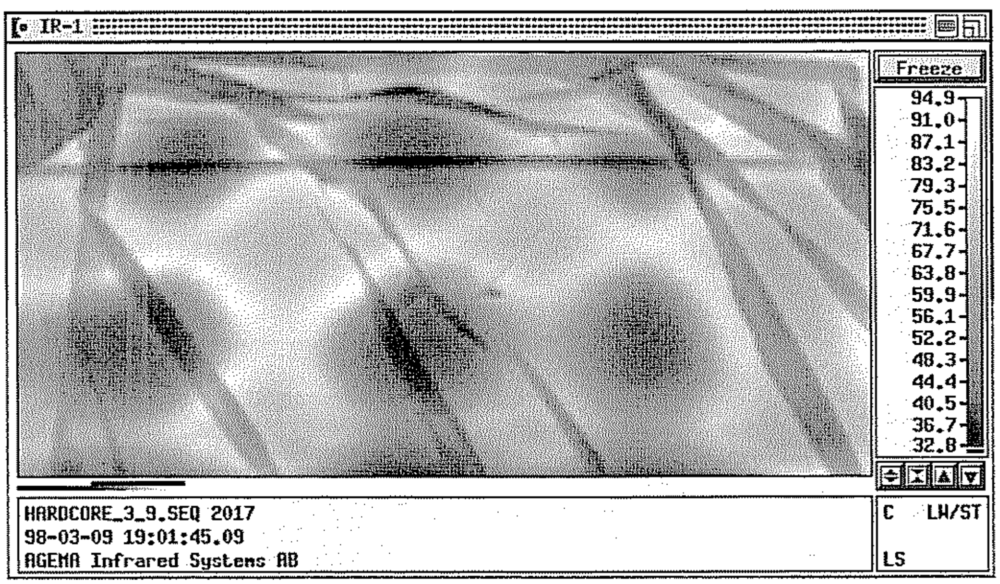
Thermocouples have to be embedded at various locations in the thick-section area with high exothermal potential to ensure maximum allowable processing temperature. Figure 4 (a) shows the temperature development in an 18 in. x 24 in. x 1 in.-thick E-glass/vinyl ester VARTM part at various locations through the thickness of the specimen. Here, the peak temperature during cure is the highest in the center of the part. Lower temperatures are obtained on the surface as well as on the bottom of the part due to the heat convection to the tool and air. Heat profiles taken during the FRP Bridge 1-351 impregnation and cure process have shown peak temperatures in excess of 220°F reaching the maximum allowable processing temperature Figure 4 (b).

2.3 Implementation of inspection requirements

As part of VARTM processing of sandwich bridge components, the resin flow to the intersection of the sandwich core walls and the face sheets slows prior to full absorption of the resin into the core walls and the facesheet fibers. Consequently, after this absorption is completed, the intersection may end up resin depleted. This is a critical point for maintaining the mechanical properties of the bridge component. The most effective means of inspecting typical intersection points is to remove cores from the bridge component itself. The disadvantage of removing such cores is that the area subsequently will need repair and will probably not be restored to its original properties. The tradeoffs in this inspection procedure must be considered for any bridge component design using resin infiltration processing with facesheets around a permeable sandwich core.



(a)



(b)

Figure 5: Monitoring of the temperature development due to the exothermic reaction in thick-section composite materials can detect out-of-spec manufacturing

(a) Thermocouple response of a 1-inch specimen shows exothermic reaction

(b) Surface temperature distribution during fabrication of Bridge 1-351 shows maximum temperature around foam cores in area with highest resin density

2.5 Bridge 1-351

The most important QA/QC techniques on Bridge 1-351 were

- Injected resin volume – the required volume of resin based on the final thickness
- Vibration technique

The effect of limits on dimensional control were minimized by fabricating and placing the deck and then pouring the supports around it.

3. Review of Composite Bridges

3.1 FRP composite materials

The use of steel or steel-reinforced concrete can lead to failure and/or costly repairs attributed to accelerated corrosion and fatigue of the materials. This problem requires new corrosion-resistant materials that can be fabricated in large sections at competitive costs to traditional materials. Composite materials have been broadly used in aerospace, defense and marine applications and they also are gaining popularity for civil infrastructures applications because of their comparable performance and projected lower life-cycle cost for maintenance (Sierakowski, 1999). Acquisition costs remain high compared to traditional construction materials. However, the economics of FRP for rehabilitation of existing bridge components can be more favorable. The stiffness and the load capacity of a degraded member are restored to values close to their original ones with the use of commercially available graphite pultrusion. Crack propagation has been drastically reduced by the adhesion of graphite pultrusion (Ammar, 1996). A Brazilian bridge, for example, has been rehabilitated with carbon fiber and epoxy resin at a 30- to 40-percent cost reduction (Horizonte, 1999).

Composite materials are worthy candidates for consideration in many critical structural elements. FRP beams and decks are currently being explored for bridge construction. FRP wrapping also is being considered to enhance the strength of concrete beams and columns, both for new construction as well as the rehabilitation of existing structures (Howie, 1996; Halabe, et al., 1999). These lightweight materials offer the capacity to decrease the overall dead load of the bridge, thereby resulting in a decrease in the required substructure to support the bridge. In addition, surface treatment or painting is not required to prevent corrosion of the composite structure because these materials also have a high degree of chemical inertness to most civil engineering environments. These merits strongly suggest their consideration for bridge infrastructure renewal. Appropriate materials include carbon and glass fibers combined with thermosetting epoxy, vinyl ester or polyester resins. Fabrication processes to combine the fiber and matrix include resin infusion, wet lay-up, pultrusion, RTM, VARTM, SCRIMP, and hybrid approaches (Eckel, 1998).

Barriers to further use of composites for bridges are initial high cost, the lack of readily available standardized design methods and the lack of standardized NDI techniques for QA/QC and field inspection. Consequently, FRP bridges are typically demonstration projects (Hooks, et al., 1997). Nevertheless, composite materials are the most promising and potentially useful materials available for bridge applications. Their merits include that they are lightweight, resistant to chemical attack, require minimal maintenance, are easy to replace, and possess superior damping properties, integral fabrication, and high strength and stiffness (Demitz, 1999).

3.2 FRP bridge 1-351 in Delaware

The Delaware Department of Transportation (DelDOT) is responsible for inspecting approximately 1,250 bridges. Table 2 shows the number of bridges in Delaware by primary material and structural design as given in the Delaware bridge inventory (Edberg, 1994).

Table 2 Delaware bridge inventory

Structural design \ Material	Concrete	Steel	Pre-stressed concrete	Timber	Masonry	FRP composite	Others
Slab or deck	273			23		2	
Stringer or girder	3	334	10	37			
Tee beam	5						
Box beam or girder		3	82				
Frame	53						
Truss		15		1			
Arch	38	8			7		2
Suspension		2					
Movable		11					
Culvert	203	116					23

The application of FRP to bridges has become a major research topic at UD-CCM to address deterioration of the aging bridges. There are two composite bridges on the state inventory (Bridge 1-351 and Bridge 1-192-318C). Bridge 1-351 was one of the first state-owned, all-composite bridges and was designed and fabricated in the fall of 1998 by a team of University of Delaware, industry, and DelDOT engineers. Figure 6 shows a picture of Bridge 1-351 during installation.

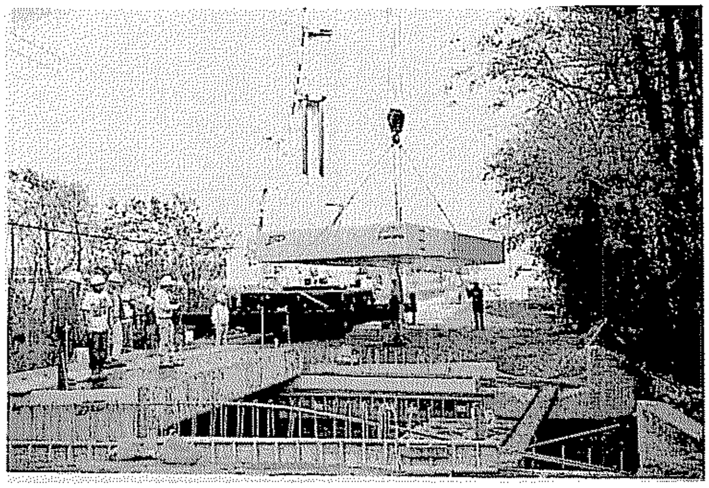


Figure 6 Bridge 1-351 during installation.

The bridge was designed to be a self-standing simply supported span. The completed bridge dimensions are approximately 31 inches high by 26 feet wide by 32 feet long in the span direction. The sectional geometry consists of a web-core sandwich construction with two E-glass facesheets (See Figure 7). To ease transportability, the composite bridge was manufactured in two 13 feet wide by 32 feet long sections. Each of these sections weighs about 14,200 pounds, which represents a 90 percent weight savings compared to traditional concrete and steel construction. The two sections were butted together and adhesively bonded using splice plates on the top and bottom of the deck section. The splice plates are approximately 0.25 inches thick, 32 inches wide and 32 feet long and are attached in the span direction. The longitudinal joint is structurally redundant and was designed with comparable stiffness in the longitudinal direction. Live and dead loads are transferred through the joint by shear of the 0.5 inch vertical joint and flexure through membrane action of the splice plates. After the sections were positioned, a 1.75-inch thick latex modified concrete (LMC) wear surface was applied to the deck surface. Parapets (crash barriers) made of concrete were placed adjacent to the FRP bridge on self-supporting precast concrete girders. Drainage was handled by providing a 2 percent gradient, running from the bridge centerline to each edge.

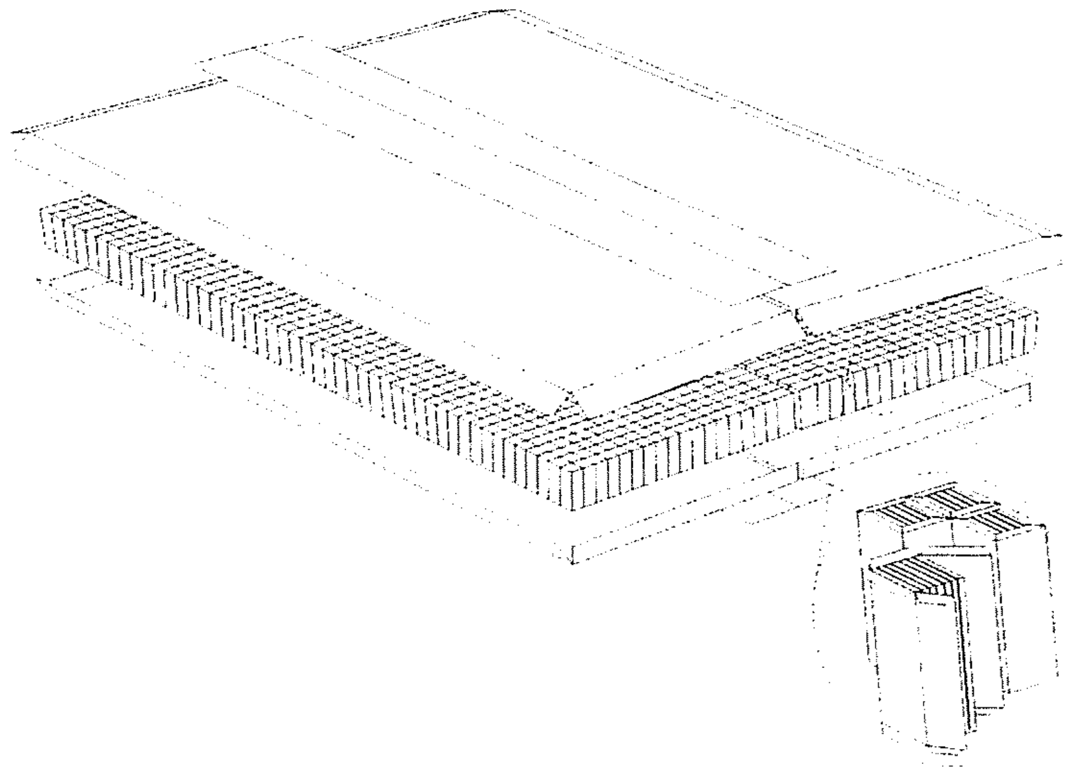


Figure 7 Schematic design: placement of bottom facesheet, wrapped cores and top facesheet. The splice plates are adhesively attached on-site.

The bridge was fabricated by Hardcore Composites of New Castle, Delaware, using E- glass fabric and vinyl-ester resin. Fabrication processes included placing QM6408 pre-cut fiber preforms in the desired geometric location and infusing them with vinyl-ester resin using a vacuum-assisted resin transfer molding (VARTM) process. The patented infusion process used was SCRIMP® (Seemann Composite Resin Infusion Molding Process). The QM6408 fiber preform consists of five individual layers (0°, $\pm$45° and 90° orientations) knitted together. The fiber preforms had a total thickness of 0.088 inches and were stacked to achieve the desired thickness to satisfy stiffness and strength requirements. To minimize costs yet retain structural capacity, the QM6408 fabric was used for both the facesheets and in the core. The sandwich structure was fabricated with a bottom facesheet 0.7 inch thick and a top facesheet 0.5 inch thick. The facesheets of each bridge section were separated by a core consisting of 893 foam rectangular prisms. The 8.125 inch x 8.125 inch x 28 inch high prisms were wrapped with one 0.088-inch thick layer of the QM6408 fabric. Adjacent wrapped prisms formed one web, with a total web thickness of 0.18 inch. Over the abutment seat, the rectangular prisms were "double-wrapped" to prevent web buckling. This resulted in a total web thickness over the abutment of 0.352 inch. The 8-fiber preform layers (40 plies) of the bottom tension facesheet were placed against a tooled surface and the prisms comprising the core were placed next. Cells modified for lifting-lugs and anchor bolts replaced standard cells. The 6-fiber preform layers (30 plies) comprising the top compression facesheet were placed last. The entire section was sealed with a vacuum bag and was evacuated. Once evacuated, the liquid resin was allowed to flow or infuse into the dry fiber preform. The resin was infused at room temperature and cured at elevated temperatures due to chemical processes. It took a five-person crew approximately 10 working days to manufacture each section.

Various NDI techniques were tested during the fabrication. For example, embedded fiber-optic sensors showed great promise for capturing the changes in internal strain due to the residual stress during cure. Infrared inspection of the bridge sectors during infusion and cure also showed promise. Several field projects are in place for long-term health monitoring such as static, dynamic live load tests, and vibration tests. The monitoring includes initial diagnostic load testing to characterize the as-built bridge behavior and a NDI program to help identify any changes in the bridge's condition over time due to sustained loads, traffic loads and environmental exposure. The initial load test provides baseline data against which the results of future load tests and the long-term monitoring data are compared for indications of any change in the bridge behavior.

The long-term monitoring program has been developed to record, store, and report key parameters that include traffic statistics, deck strains and deflections, girder rotations and weather conditions. The bridge carries very limited truck traffic in terms of live loads. Testing conducted at the University of Delaware has indicated that 2,000,000 fatigue cycles will cause negligible losses in strength and stiffness to the GFRP deck (Chajes, et al., 1998). With these data, deflection, strength, and fatigue-limit states of the bridge can be monitored over an extended period. By utilizing the collected field data in conjunction with results from laboratory tests, the future life of the bridge can be reliably projected (Chajes, et al., 1997).

4. Defects and inspection areas of bridges

4.1 Composite structures

Defects in composite structures occur during fabrication or during service life. Defects during fabrication include cracks, delaminations, disbond, porosity (dry spots), core crush and foreign material contamination. Cracks are regions of actual separation of the material, visible on opposite surfaces of the part, and extending through the thickness. Delamination is the separation of the layers of material in the laminates cured in the cross-section. Disbonds can be physically separated by an air gap or can be in intimate contact (the latter being particularly difficult to detect using NDT). Disbonds in FRP laminate are normally considered delamination. Fatigue loading can cause delamination propagation and can lead to stiffness loss and structural failure. Porosity is a condition of trapped pockets of air, gas or vacuum within the FRP. Porosity (or voids) is a typical defect that causes a significant reduction in the matrix-dominated properties of the composite. Porosity acts as a stress concentrator that can lead to failure at low load levels. It also can act as a pathway for the diffusion of moisture and air into the system, which can cause degradation of the fiber/matrix interface, plasticization of the polymer and the acceleration of freeze-thaw damage. Void volume fractions must be kept at minimal levels in high-quality composite parts (Fecko, et al., 1996). Foreign materials can be an inclusion of a foreign substance during the manufacturing process that adversely affects the local performance and limit life of the FRP (Register, et al., 1998). Contamination (physical and chemical) can significantly degrade the stiffness and strength of composite structures. Core crush is a condition when core has been damaged during fabrication or overloaded during erection and cure. Material properties are very dependent upon process conditions such as temperature, pressure, flow rate, and degree of cure. Ideal QA/QC should incorporate NDI techniques for the maintenance and control of the in-coming materials and the manufacturing process and cure of the components to minimize defects during fabrications.

Defects during service life include matrix cracks, delamination, fiber breakage, fiber matrix debond, impact damage, core crush and fracture. Initiation and growth of defects can continuously degrade the strength and stiffness of the structure over its service life. The typical service conditions in a bridge are categorized as follows:

- Environmental conditions

- Sustained loads
- Cyclic loads
- Handling or impact damage

Environmental conditions include moisture, salts and freeze/thaw. Moisture is absorbed into the matrix resin due to the attraction of water molecules to polar molecules within the resin, which weakens the hydrogen bonding between polymer chains. Swelling of the matrix may cause cracks resulting in a reduction of strength. Stiffness is reduced through moisture-induced plasticization. Since glass and graphite fibers do not expand with moisture exposure, the expansion of the resin due to water absorption can lead to a stress discontinuity at the fiber matrix interface, resulting in fiber matrix debonding. For example, E-glass/polymer systems had a 35% reduction in tensile strength and failure strain after 417 days in 100% at 38oC. However, they were not affected greatly by salt water or alkali solution. In addition, they had no significant reduction in Young's modulus from environmental exposures (Steckel, et al., 1999).

When dead loads are significant or the component is subjected to a significant pre-stress, creep can occur. Creep corresponds to the rearrangement of polymeric molecules that results in the occupation of the free volume spaces in the aging composite. Creep is greatly accelerated by temperature and thus is, in particular, important for components subjected to extreme temperatures, such as decks under tropical sunshine. Creep can be ignored for short-duration loads, such as vibrations, shock loads and live loads and in bridges with minimal dead load, which is typical in FRP bridge design. (Demitz, et al., 1999).

Cyclic loads below ultimate result in fatigue: the accumulation of damage. Fatigue in composites is characterized by the accumulation of micro cracks, while the fatigue in metals consists of initiation and growth of a single crack. The development of fatigue damage in unidirectional composite laminates may be broken up into two distinct stages. The first stage consists of non-interacting cracking restricted to individual plies. Matrix cracks running transverse to the loading will form in off-axis plies. Figure 8 shows transverse matrix crack through the 90^{o} layer laminate due to fatigue load (Morin, 1993).

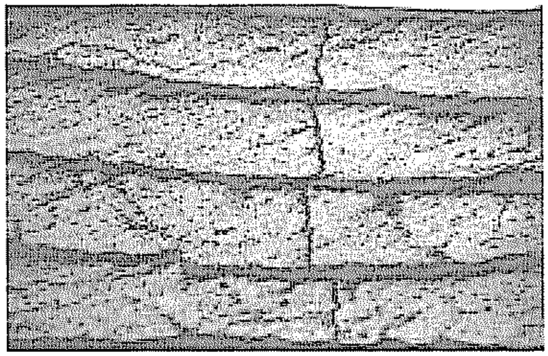


Figure 8 100x photograph of the cross-section of an X5260/G40-800 [$0_2/90_4/0_2$] laminate showing a full 90° layer thickness crack

Stress concentrations at the crack tips cause fibers in adjacent plied to break, resulting in a redistribution of load through the composite. The second stage of damage is characterized by the formation of cracks parallel to the load direction in on-axis plies. Parallel cracks will interact with the transverse cracks forming local ply delaminations. The transition between the first stage and second stage of damage is the characteristic damage state (CDS). For off-axis angles of fatigue loading, the tip of a crack will have two displacement components: opening normal to the fiber and sliding parallel to the fiber. The opening mode of crack growth is the more critical of the two. Most laminates consist of a sequence of plies with differing reinforcement angles, such as 0°, 45°, and 90°. The failure sequence has been found to be failure of the 90 plies followed by the 45 plies, leading to overstressing of the 0 plies and consequent failure. However, this may not be true for composites containing plies that are woven fabrics (Demitz, et al., 1999).

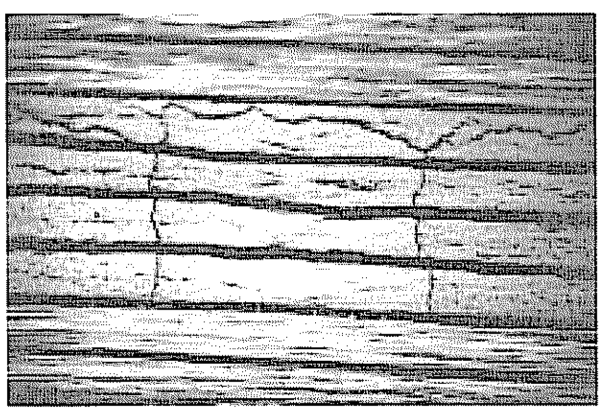


Figure 9 100x photograph of the cross-section of an X5260/G40-800 [$0_2/90_4/0_2$] laminate showing development of longitudinal cracks after near-saturation fatigue loading.

After initial transverse cracks have formed, the cracks grow and the crack density increases. Finally, the longitudinal cracks occur, prior to the saturation of the crack density. The longitudinal cracks are believed to be evidence of delaminations initiation. Note that the delaminations in Figure 9 originate at the tips of transverse cracks where interlaminar stresses exist. The damage process of composite laminates subjected to monotonically increased loads is the same for laminates subjected to tensile fatigue loads. Figure 10 shows the growth of damage over the life of a unidirectional composite laminate. With increased fatigue cycles, the delamination would be expected to propagate resulting in stiffness and strength reduction at premature failure (Morin, 1993). Therefore, FRP structures should be designed in such a way that the microcracks are not critical (lower stress/strain levels below fatigue run-out). This class of damage is a challenge to detect by NDI.

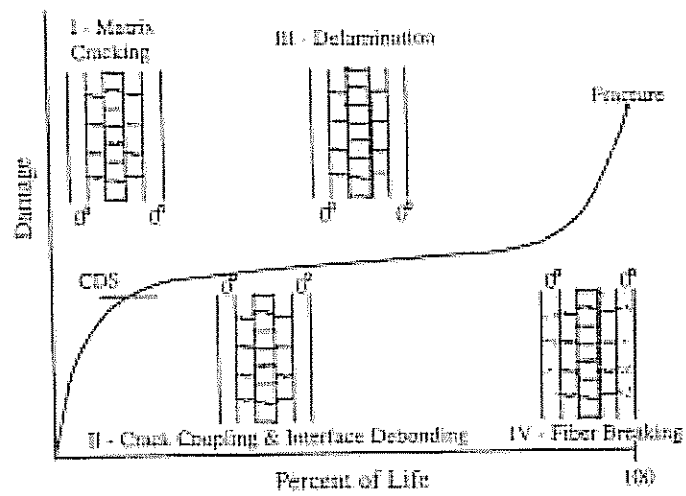


Figure 10 Development of fatigue damage in unidirectional composite laminates.

Damage may be incurred during transportation and installation of the component, by acts of vandalism, or by extreme service conditions. Impact damage to the bridge may be caused by overloaded trucks, vehicles traveling over the speed limit, or by traffic accidents. In addition, another potential source of impact damage is tree branches hitting the bottom of a FRP bridge.

Consideration should be given to the failure modes of composite structures in order to understand damage mechanisms. Failure of a composite when properly designed is normally a progressive sequence of events because composites are internally redundant. For example, a composite sandwich subjected to bending may have several failure modes, such as overstressing of the panels, buckling of the compression face, core shear failure, interlaminar shear failure of the panels, buckling of the compression face, core shear failure, interlaminar shear failure between the core material and face sheet, or transverse shear failure of the core. A composite plate bonded to the tensile flange of a steel girder may fail by: overstressing of the reinforcing fibers, shear failure in the adhesive bond line, or peeling from the flange due to excessive curvature. For composite laminates consisting of multiple-ply orientations subjected to tensile static loads, the material response exhibits a gradual failure beginning with initiation of matrix cracking in one or more plies as the first critical event. The last critical event is the fracture of the laminate resulting from fiber failures in the plies most closely aligned with the principal tensile stress. The compression panel can be overstressed, resulting in cracking of the matrix and ply delaminations, provided the panel does not buckle. The sources of damage will be reflected on the mechanical properties of the member, such as stiffness. The event of first-ply-failure is the critical point at which damage initiates in a composite. In addition, the stiffness is a potential NDI parameter, which could be used to monitor the damage in a component while in service. Therefore, damage is defined as the point at which the mechanical properties of material have been altered as the materials continue to be loaded (Demitz, et al., 1999). These mechanical properties are obtainable by solving inverse problems with vibration techniques.

4.2 FRP Bridge 1-351 in Delaware

The Bridge 1-351 has a web-core sandwich deck construction (Eckel, II, 1998). A schematic diagram of FRP sandwich deck is shown in Figure 11 to describe various possible damage modes. Live and dead loads generally cause a flexural bending moment in the bridge, which produces compression forces in the top facesheet, tension forces in the bottom facesheet, and shear forces in the web-core layers. Shearing of the interfaces between facesheet and core layers is a critical mode of failure that causes separation and delamination of the core from the facesheet.

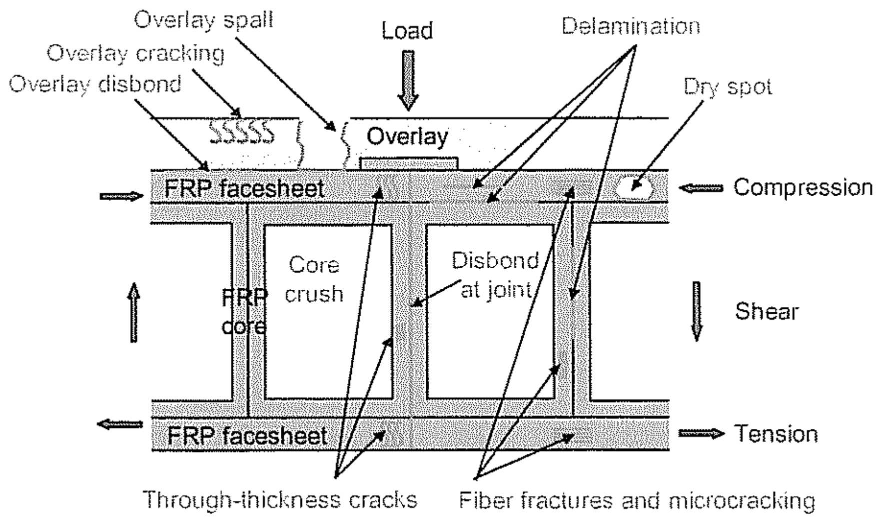


Figure 11 Schematic diagram of defect types in FRP sandwich deck.

Transverse shear forces also may induce interlaminar shear failure at the intersection of the facesheets and the core, as shown in Figure 12 (Eckel, II, 1998). Delamination and propagation of damage will result in separation of the facesheet and core. This will result in significant loss of stiffness and load capacity.

Figure 12 Side view of the deck section showing delamination of the top facesheet from the core.

Crushing of the core occurs when the core does not have a large compressive capacity relative to wheel loads. When the sandwich is subjected to transverse loads, shear forces may induce core shear failures. Large transverse shear deformations can lead to finite rotations at the intersection of the core and the facesheets, inducing a peeling type of fracture. Subsequent delamination and crack growth during fatigue results in the loss or decrease in horizontal shear capacity and ultimately to catastrophic failure. Figure 13 shows buckling or crushing in a web-core cell at the failure load. Note that the majority of the failed or buckled areas appear to be near the top of the specimens (Eckel, II, 1998).

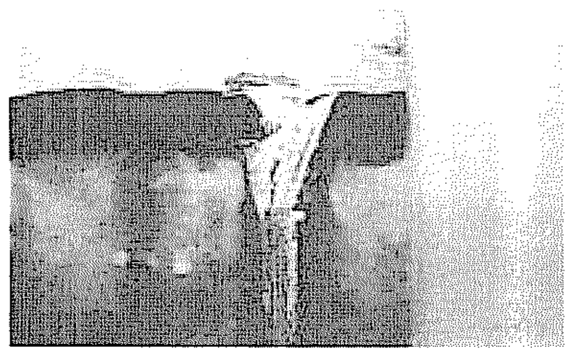


Figure 13 Crushing of a web core between the top facesheet and the web core at the failure load.

Facesheet buckling or wrinkling may occur near load introduction or when the facesheet is structurally too thin. When subjected to compressive forces, thin webs may result in a loss of stability. Web buckling is another failure mode that may lead to cracking or delamination. As is the case with most sandwich structures, shear failures initiated by peeling at the interface of the facesheets and the core is the predominant type of failure for the web-core deck sections (Eckel, II, 1998). Cracks and spall in the overlay are visible and hence can easily be repaired. However, the other defects in the facesheet and web-core layers need proper NDI techniques to determine the location and size of the defects. Water absorption causes accelerated freeze/thaw change and decreases the global stiffness of the deck as the bridge ages. Therefore, global NDI techniques are required, such as a vibration technique. Table 3 shows the defect modes and their locations for the FRP sandwich deck of Bridge 1-351.

Table 3 Defect modes for the FRP sandwich deck of Bridge 1-351.

Generation Modes	Defects	Location	Cause	Recommended NDI
Fabrication	Delamination	1. Between face sheet and core 2. In the facesheet layer 3. In the web core	Contamination Poor cure	Vibration Ultrasonics
	Dry spots	1. In the facesheet layer 2. In the web core	Poor resin infusion	Vibration Ultrasonics
	Disbond	Joint	Poor installation Handling	Vibration
Service	Delamination Buckling Wrinkling	1. Between facesheet and core 2. In the facesheet layer 3. In the web core	Fatigue Overload Thin facesheet	Vibration
	Transverse cracks	1. Facesheet 2. Web core	Fatigue Overload	Vibration Ultrasonics
	Fiber fracture	1. Facesheet 2. Web core	Fatigue Overload	Vibration Acoustic Emission
	Core crush	Web core	Over compression Thin-web core	Vibration
	Disbond	Joint	Fatigue Overload Impact	Vibration
	Creep	Global area	Environment (water)	Vibration
	Spall	Overlay	Environment Impact	Visible

Some damage may be negligible, but other damage may be critical to the future safety of the bridge. Analysis and experimental studies to determine defect criticality may be required. However, NDI data (defect type, size and location) is a critical input to bridge rating. Therefore, simulation and experimental studies are required for the effects of the damage on the properties and future lifetime of the deck. All these studies can be done by several emerging NDI techniques.

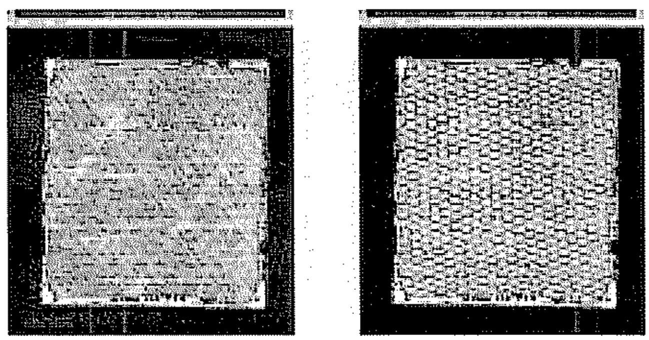
5. NDI applications for composite structures

5.1 Property tests in laboratory

Various laboratory techniques are used to determine physical properties such as density, fiber volume fraction, and thermal expansion coefficient. Mechanical property tests include tensile, shear strength, bending, compression, fatigue, and fracture tests. The test methods and procedures are well defined as various ASTM or ISO standards. Frequently, specimens can be examined with a scanning electronic microscope (SEM) or an optical microscope. High-quality materials should have minimal void content/porosity (<1-2%), no delamination and fiber alignment in close agreement to design specifications.

5.2 NDI in laboratory

Laboratory level tests for NDI are widely performed. NDI methods of composite materials include visual, tap testing, ultrasonic, acoustic emission, acousto-ultrasonic, radiography, thermography, laser shearography, and eddy current inspections. Visual inspection can detect defects such as edge delaminations around drilled holes or on the edge of the panel. It also may detect porosity, crack, core crush, disbonding, or cut fibers. Visual inspection of composite materials is equally as important as other methods and should be the first step utilized before performing other methods. Tap testing also can detect delaminations, porosity, core crush, and cracking. Generally, tap testing provides preliminary information. Tap testing should be followed up by another NDT method, such as ultrasonic inspection (Register, 1998). Ultrasonic technique (C-scan) is widely used to inspect flaws in composite panels. It can routinely detect delaminations, porosity, and foreign material contamination when the size is greater than 0.25". Bond quality is tested with a lower frequency ultrasonic method. Air-coupled ultrasonic inspection is used for measuring foam structures with a lateral resolution of about 0.040" due to the focusing effect of the air-coupled transducers (Strycek, et al., 1996). Figure 14 shows an example of a UT image for a honeycomb sandwich structure. Figure 14 (a) illustrates the surface image and Figure 14 (b) shows the image of the internal structure. Water was used for coupling the UT sensors and the specimen. Porosity gradient as well as thickness variation of polymer matrix composite are determined by the ultrasonic contact scanning method (Roth, et al., 1995). Impact damage is evaluated by analyzing the elastic wave, which is introduced into composites by an ultrasonic testing probe (Ozaki et al. 1992).



(a) surface (b) core

Figure 14 Ultrasonic image of a honeycomb structure

AU techniques can be used to effectively identify the bond strength, the transverse tensile strength, and progression of damage in transverse specimens (Quattlebaum, et al., 1997). AE is used to inspect transverse failure, which is an important damage mechanism in controlling the initiation of damage in a composite. Therefore, an AE provides a unique view into the micro-mechanics of defect initiation and growth (Hill, et al., 1998). Scanning electron microscope (SEM) is applied to identify the topographic features of failed and fatigued graphite/epoxy coupons (Morris, et al., 1982). Holographic interferometry using thermal loading can accurately track delamination growth, but cannot find subsurface matrix cracks (Maddux, et al., 1979). Of the available NDI techniques, only penetrant enhanced X-ray radiography has the resolution capability to record matrix cracks, delaminations, and fiber bundle fractures. Unfortunately, it does not provide through-the-thickness information. To overcome this deficiency, a penetrant-enhanced stereo X-ray method has been developed to construct three-dimensional images of damage in graphite/epoxy specimens (Sendeckyj et al. 1982). Scanning refractometry with X-ray scattering techniques visualizes interface properties at 10 m spatial resolution by computer image. Applications include ceramics and impact damages on CFRP-plates (Harbich et al. 1996). Thermography is used to detect large delaminations or disbonds that are near the surface in ply depth. However, cracking and porosity are difficult to identify with thermography. A vibration technique can be used to identify damage location (Cawley, et al,. 1979, Ratcliffe, et al,. 1998), and microscopic damage location (Boxwell, et al., 1997). Damage is localized by the comparison of the sensitivity matrix of the undamaged and damaged structure (Crema, et al., 1995). The interfacial adhesion between fiber and matrix in fiber-reinforced composites is evaluated by modal damping measurements (Visscher, et al., 1996). Laser-induced thermal shock produces elastic waves in composites; fiber-optic interferometry then provides the non-contact point measurements of the resulting displacement-time histories (Burger 1993).

5.3 NDI during manufacturing

In order to improve the overall structural performance, quality control is necessary to identify, locate, and analyze defects. NDI that measures the quality of the composite on-line can be combined with intelligent control units in order to develop an automated manufacturing system. An added advantage of in-process inspection is that it eliminates the need for post-processing inspection. Most importantly, inspections of composites in the field enable defects to be repaired before the construction of structures is completed.

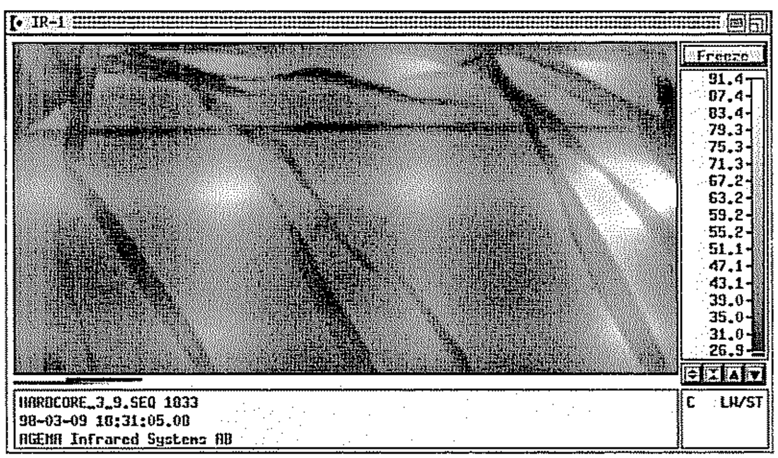


Figure 15 Resin infiltration during fabrication of Bridge 1-351.

The University of Delaware Center for Composite Materials (UD-CCM) has developed the application of acousto-ultrasound for detection of void content and stiffness changes (Fecko, et al., 1996), as well as a new laser-based technique for the detection of ultrasonic waves in solid materials—Gas-Coupled Laser Acoustic Detection, known as GCLAD (Caron, et al., 1997). In addition, an infrared thermography system for online temperature measurement has been shown to be sensitive to cure and delamination detection. Figure 15 shows the IR image during fabrication of FRP Bridge 1-351 deck, where high temperature regions illustrate the curing of resin that flows around the core cells.

UD–CCM researchers are investigating several innovative online-sensing techniques and applying them to core manufacturing areas for the composites industry. These include SMARTweave, a distributed network of direct-current sensors that is used to monitor mold filling and cure during RTM and VARTM; and fiber-optic sensors, which are being used for in-situ monitoring of cure, shrinkage, and warpage, as well as for health monitoring of structures. SMARTweave can be used to inspect all regions of the perform that are fully impregnated. Fiber-optic sensor methods can be applied to measure internal strain and thus residual stress during fabrication. Successful implementation of SMARTweave, embedded fiber-optic, and infrared thermography equipment are critical sensors to provide feedback of part quality. A number of other sensing techniques to determine fabrication quality and part performance and to promise NDI capabilities for large-scale structures are currently being implemented.

5.4 NDI field inspection for composite bridges

New designs and materials represent new challenges for NDI of large-scale composite bridges. Even though the primary means of inspecting bridge components is visual, the visual appearances of composite materials may belie actual condition unless the deterioration is quite severe. Furthermore, delaminations between layers due to manufacturing or impact damage are most visible from the surfaces.

Ultimately, this challenge may be resolved by using incorporated or embedded sensing systems, or through the use of proper NDI methods (Duke, 1998). The use of advanced sensor technology will yield a wealth of information about the performance of the bridge under actual service conditions. This feedback will enable optimization of future deck designs. Several sensors can be suggested for the composite bridge application, such as fiber optics, time-domain reflectometry, strain gauge, and piezoelectric sensors. UD-CCM utilized various embedded sensing techniques on the 1-351 all-composite bridge. Prior to the bridge construction, fiber-optic sensors were used on the large composite parts to test the structural properties. The embedded sensors have survived without damage after construction of the bridge.

Thermography was utilized for the inspection of composite seismic retrofits to concrete structure. While the initial equipment costs are high, the advantages of thermography include increased sensitivity, remote inspection capability, decreased inspection times and convenient means for data archival (Hawkins, et al., 1999). For Bridge 1-351, IR techniques are: inspecting delamination of the facesheet and the facesheet core, as well as debond of overlay from the facesheet. It is difficult to detect damage in the core, but IR imaging was successful in providing an overall assessment. UT imaging was not successful due to the large level of attenuation for the pultruded vinyl ester double box I beam reinforced with glass and carbon fibers (Duke, 1998). UT, however, can be used to detect surface facesheet delamination or internal core crushing of composite bridges if air couplant is used and proper scanning methods are developed. The AU technique can be used as a destructive test for the QA purpose of FRP panels. It could be used for a field test of Bridge 1-351 by developing exciters with high energy and sensors with low frequency bands, and by using air couplant sensors. However, vibration techniques would be a better choice for the exciting and sensing of large-scale composite structures such as FRP bridges because NDI techniques are sought to enable identification and evaluation of the "global" condition or health monitoring of highway structures. AE techniques can be used for monitoring the occurrence of initial transverse cracks during tension and fatigue-load condition. In addition, AE techniques may allow better determination of the location and depth of cracks through the use of three or four AE sensors. Radiography may be used for QA of large FRP structures before installation as long as scanning is possible over a large area of the bridge. However, it is virtually impossible to utilize field inspection because the image of the radiography can be obtained only on opposite side of the bridge deck. MPI or ECT cannot be used for traditional glass-fiber composite structures because these techniques require the samples to be magnetized or conductive, respectively. GPR may be used for detecting the internal geometry variation of the FRP bridges, such as core crushing. TDR may be used for the delamination monitoring of the FRP bridges. These NDI techniques have to be quantitative, comprehensive and sensitive to a variety of types of damage that may occur during the life cycle.

Load tests were employed to monitor Bridge 1-351 for performance. The static testings do not use destructive methods so thus they may be categorized as NDI techniques. The bridge rating also was used for designing FRP Bridge 1-351, and the load test and impact test were executed to evaluate the performances of the FRP Bridge 1-351. Note that the vibration NDI technique is the only method available to evaluate large composite structures locally as well as globally. It correlates global stiffness of a structure to natural frequencies and local defects to processed mode shapes. It is cost effective and easy to operate, and has the potential for online damage detection with appropriate structural modeling. UD-CCM performs annual field vibration tests on the 1-351 all-composite bridge in order to assess global bridge properties, including structural integrity, as well as to detect localized damage (See APPENDIX E).

6. Assessment and recommendation of NDI techniques

6.1 Assessment of current NDI techniques in civil engineering

This chapter summarizes the previous information and presents recommendations of NDI techniques. Table 4 summarizes the applications and characteristics of common NDI techniques used in civil engineering, which include visual and optical method, thermal method, ultrasonics, acoustic emission, acousto-ultrasonics, static and dynamic diagnostic method, radiography, liquid penetrant inspection, magnetic particle inspection, eddy current method, ground-penetrating radar and time-domain reflectometry. Many advanced sensing and measurement technologies have been developed or transferred from defense use to infrastructure applications in the past decade. Many of the NDI methods are highly sophisticated, however, there also are techniques that are relatively simple. One such a method is visual examination, which is both simple and always applicable. NDI may be combined with those destructive techniques that can be performed in the laboratory on samples removed from the structure. Thermal inspection techniques such as IR image method can be used to evaluate concrete structures such as decks and girders by fast scanning of the large area, but it possesses inherent problems, such as uniform heat source treatment. Ultrasonics, which are the most widely used NDI technique, possess such limitations as requirement of coupling media, difficulties in field application and limits on penetration depth. This is why its application is confined to laboratory experiments. AE is especially useful for progressive damage, such as crack propagation, but it can be difficult to estimate the severity of damage when using AE. Static load tests are the most popular method used by state department of transportation offices to identify the rating of bridges with strain instruments. Static load tests are usually performed with live load tests to get dynamic deflection. Vibration technique has been widely studied because of its tremendous potential, which will be discussed in the next subsection. Radiography is useful to detect internal defects of thick specimen, however, it is not easy to perform because of portability issues. TDR is useful for detecting cracks in the cables of suspension bridges and GPR is useful for inspecting bridge decks. GPS is another emerging technology that can identify the movement of bridge substructures, such as piers and abutments.

Table 4 NDI techniques in civil engineering

NDI Technique	Methods	Applications	Characteristics
Visual or Optical	1. Video image 2. Holography 3. Interferometry 4. Liquid penetrant method	Every accessible part	Detection of surface defects
Thermal Inspection	1. IR image method 2. Liquid crystal compound 3. Temperature-sensitive paint	Concrete structures	1. Quick identification 2. Remote 3. Portable 4. Minimal disruption to traffic 5. Scans large area
Ultrasonics	1. Pulse echo 2. Pulse transmission 3. Reflection 4. C-scan 5. DSSSUE	1. Crack detection in concrete structures 2. Quality in bridge weld 3. Bridge-pin integrity 4. Corrosion in steel bridge	1. Pulse echo for local 2. DSSSUE for global
Acoustic Emission	DAME AEWM	1. Fatigue crack in steel bridges 2. Steel lock 3. Dam	1. Easy monitoring with remote sensors 2. Highly sensitive crack growth 3. Stabilized crack can not be detected 4. Size of crack can not be detected
Acousto-ultrasonics		Distributed damage such as bridge deck	Characteristics of both ultrasonics and acoustic emission
Static load test	1. Static diagnostic testing 2. Proof testing	Bridge	Used for bridge rating with deflection or strain instruments
Vibration	1. Impulse 2. Harmonic 3. Traffic induced	1. Bridge 2. Dam 3. Continuous monitoring	Global bridge monitoring with frequency response function, operating deflection shape and modal parameters
Radiography	1. X-ray 2. Gamma ray		1. Local inspection 2. Radiation emission
Magnetic	1. Magnetic particle inspection 2. Eddy current method	1. Nuclear reactor by EC 2. Fatigue crack by MPI 3. Stress gradient in steel railroad bridge girders 4. Bridge cable by MPI	Ferromagnetic material such as steel
Ground Penetrating Radar	1. Impulse radar 2. GPR	1. Bridge deck inspection 2. Pavement inspection 3. Soil water content detection	In-situ inspection mounted on a service car
Time Domain Reflectometry		1. Steel cables 2. Mine detection 3. Frazil ice	

1. Table 5 reorganizes previously mentioned NDI techniques according to development phase (e.g., research, developments, field ready) and damage types. Some NDI techniques such as portable ultrasonics, static load rating tests, and vibration techniques are widely used and ready for field applications, while most NDI techniques are in developing or research stages.

Table 5 Development of NDI techniques in civil engineering

NDI Technique	Development Phase	Performance Level	Damage Types	Group by Coverage
Visual or Optical	5	III	All visual surface defects	Local inspection
Thermal Inspection	1, 2	II	Hidden deterioration in concrete slab	Local, large area by scanning, remote monitoring
Ultrasonics	3, 4, 5	II, III	Internal flaw, Crack in connection or weld	Local, medium area by scanning
Acoustic Emission	1, 2	II	Active corrosion, fatigue crack	Local, global, remote monitoring
Acousto-ultrasonics	1, 2	II, III	Distributed damage in deck	Local, remote monitoring
Static Load Test	3, 5	I	(Strength and stiffness)	Local, remote monitoring
Vibration	1, 3, 5	III, IV	Structural degradation	Global, local damage location and sizing
Radiography	2, 3	II	Embedment in concrete, crack in steel	Local, small area by imaging
Magnetic	2, 3	II	Subsurface defects in deck	Small area
Ground Penetrating Radar	1, 3	II	Corrosion in steel bar in concrete decks	Large area by scanning, damage location and sizing
Time Domain Reflectometry	1, 2	III	Defects in cables	Local but large area, damage location and sizing

Development phase: 1=Research, 2=Developed in some institutes, 3=Used in some state department of transportation offices 4=Commercialized by industry, 5=Field-ready application

This study classified NDI techniques in two groups, local and global defect detection techniques as follows:

1. Local group: the identification of localized deterioration, defects and damage in concrete or steel such as visual, thermal, ultrasonics, radiography, magnetic and GPR.
2. Global group: global characterization of a structure's condition such as thermal imaging, static or dynamic load test, AE, AU, TDR, etc.

Damage is not identified until the scanning device inspects a certain damage area for the case of local group NDI techniques, while damage occurrence or location can be monitored online by processing the time history data in the case of global group NDI techniques. While numerous NDI technologies exist for local level evaluation of civil infrastructures, only vibration technique has the potential to provide global and local characterization of a structure's condition (Aktan, et al., 1999). The global group NDI techniques such as AE, AU and TDR also are candidates for global monitoring (they do not require scanning). They are widely used for locating localized damage, sizing with remote monitoring while trying to be consistent with the BMS. In addition, both AU and AE offer means of assessing regions where access is limited by deck placement.

In addition to the local/global categorization, a refined classification method has been introduced. All the NDI techniques can be classified into one of four levels according to their performance (Sikorsky, 1999). These performance levels include:

- Level I – methods that only identify if damage has occurred.
- Level II – methods that identify if damage has occurred and simultaneously determine the location of damage.
- Level III – methods that identify if damage has occurred, determine the location of damage as well as estimate the severity of the damage.
- Level IV – methods that identify if damage has occurred, determine the location of damage, estimate the severity of damage and evaluate the impact of damage on the structure.

The local group NDI techniques as shown in Table 5 may satisfy Level I requirements; they may not detect damage until they scan the damaged area. AE can be classified as a Level I and II NDI technique but not a Level III NDI technique because it is difficult to correlate the severity of damage with the acoustic signals such as amplitude or frequency. Sikorsky (1999) notes that although the relative performance of several prominent Level II techniques have been evaluated by using experimental data obtained from a field structure, no such work has been done for a Level III or IV technique. Vibration techniques offer potential for not only Level III but also Level IV because it can obtain the material properties such as stiffness by system identification methods with a proper structural model. Vibration NDI techniques can be used as model-based techniques that examine changes in the vibration characteristics of the structure to give global information on the structure. However, most of the other techniques are non-model techniques that can provide only local information and no indication of the structural strength at a system level (Zou, et al., 1999). In addition, vibration NDI technique does not require direct human accessibility of the structure if it is combined with embedded sensors or smart structures.

Extensive use of NDI in the field for construction QC and in-service monitoring has not taken place so far because equipment is bulky and expensive, and highly skilled personnel are needed for data interpretation. However, the cost of NDI equipment is decreasing, as well as the size and weight of this equipment. NDI techniques can be used during construction and/or fabrication stages to ensure that the product meets the quality standards that are necessary for its intended performance (Halabe, et al., 1999). NDI tests provide defect detection in structural components, but it is generally difficult to translate the effects of deterioration to the residual load-carrying capacity, unless strength tests and detailed analysis are conducted. Ultimately, these NDI assessments provide the database for a bridge management system (BMS) that requires all NDI techniques to use objective and quantitative measurement index such as damage index (Aktan, et al., 1999) or condition state (Hearn, et al., 1998). Therefore, one of the issues for NDI techniques is the establishment of clear scientific standards to specify how a technique must be proven and validated before it may be promoted for use (Aktan, et al., 1999). Investment into sensing technologies such as fiber-optics and GPR. should follow a clear establishment of the circumstances for need and use of the NDI techniques, and only be considered in conjunction with a global perspective of the civil infrastructures performance, integrated asset management and health-monitoring issues.

6.2 Recommendation of NDI techniques for FRP bridges

As mentioned in previous sections, standard approaches such as visual and load testing are acceptable for composite bridges. However, non-visual defects need advanced NDI techniques such as UT, AE and vibration. Therefore, the procedure for NDI techniques for FRP bridges are recommended as follows:

1. The procedure starts from the visual inspection to check surface anomalies such as surface cuts, debonds, and impact damage.
2. Tap testing over suspicious area follows as an intermediate technique.
3. Then, advanced techniques are used. For example, global NDI techniques such as vibration are employed not only to evaluate the global stiffness of the bridge, but also to

locate local localized damage. The local NDI methods such as UT or AU scan over the zoomed-in area by the global NDI techniques to identify the type of defects. Note that vibration techniques can not only detect the locations, but also assess the severity of damage.

Table 6 shows various damage types and corresponding NDI techniques for composite structures. Usually, visual inspection should be the first option for most damage types. Ultrasonic is most widely used but its application is confined to local area scanning. Vibration and AE could be combined with remote monitoring systems. Both techniques are used to detect progressive damage. Currently, vibration technique is used as only a Level III NDI technique. Figure 16 shows an example of the vibration NDI techniques to localize damage area using FE methods. The experimental results for the FRP bridge is shown in Appendix E.

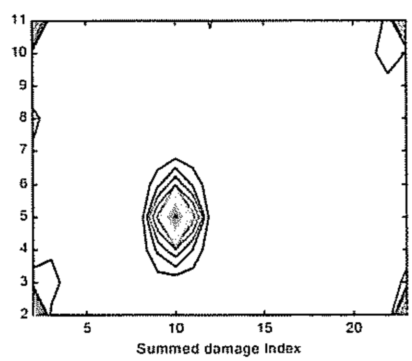


Figure 16 Contour of the damage index for 5% stiffness reduction of the local area.

In addition, vibration technique can be automated to remotely monitor structures by using piezoelectric materials for excitations. Embedded sensors such as fiber-optic sensors could be used for online monitoring to provide Level IV NDI information. To implement the online monitoring techniques, an essential condition is making the structure "smart" or the material "intelligent." This is the new generation of composite materials or structures, dubbed "intelligent material" or "smart structure" (Zou, et al., 2000).

Table 6 Damage and corresponding NDI techniques of large scale composite structures

Damage	NDI techniques	Suggested NDI (+Visual)
Crack (growth)	visual, SEM, tap, vibration, AE, radiography, EC	Vibration
Delamination	visual, SEM, tap, vibration, ultrasonic, thermography, laser shearography	Vibration
Porosity	visual, SEM, tap, ultrasonic, AU, radiography	Radiography
Disbond	ultrasonic, thermography, vibration, laser shearography,	Ultrasonic
Core Crush	tap, vibration	Vibration
Foreign Material	ultrasonic, vibration	Ultrasonic
Creep (strain)	visual, strain gauge, fiber optics, TDR	fiber optics
Impact Damage	visual, ultrasonic, AE	AE
Fracture	visual, AE	Vibration
Composite Bridge	visual, load test, fiber optics, AE, thermography, TDR, vibration, GPS	Vibration

7. Conclusion

Modern NDI methods are successfully applied in several areas of civil engineering thanks to the rapid evolution of computers and data acquisition systems, resulting in a great potential for saving time and reducing costs. These methods will enable state department of transportation offices to perform more efficiently and effectively.

There are significant gaps in fundamental structural engineering knowledge -- particularly with respect to analytical modeling capabilities -- that limit effective and meaningful applications of advanced NDI techniques. Today, there are many good career opportunities in the field, yet there is a shortage of qualified inspectors (Potter, 1999). Additional funds are needed for the training of inspectors of civil infrastructure systems. Much progress has been made in the development of promising technologies for objective condition assessment of civil infrastructure systems, but more improvements are still needed. Needed improvements include: accuracy and repeatability, easy accessibility with wireless or remote monitoring (Wisher, 1998), integration NDI method into BMS, quantitative data requirement for objective inspection with damage index and refinement adaptation of NDI techniques to civil engineering field conditions (Rens, et al., 1998).

In this report, we discussed QA/QC for manufacturing composite bridges. FRP Bridge 1-351 is introduced and defects and inspection area of are defined for composite structure as well as FRP Bridge 1-351. Several NDI techniques for composite materials are reviewed and related to the types of defects they detect. Current NDI technologies in civil engineering are investigated and their potential benefits and limitations are reviewed. Promising NDI techniques are recommended, as well. In the appendix, traditional bridge types and structure types of bridges are introduced. Critical defects are identified with respect to the traditional bridge types as well as the structure types. Basic principles of traditional NDI techniques are illustrated and the main application areas of the traditional NDI techniques in civil engineering are reviewed. Finally, an example of the vibration testing for FRP Bridge 1-351 is included.

We recommend the NDI procedure for composite bridges: visual for initial inspection techniques, tap testing for intermediate techniques and advanced NDI techniques for qualitative as well as quantitative assessment of the hidden defects. Advanced NDI techniques for civil infrastructure are ultrasonics, IR imaging, AE, GPR, TDR, and vibration techniques. If they are used together with embedded sensors, the products will meet the quality standards during construction and/or fabrication stages of advanced composite materials. After assessing current NDI techniques, we suggested vibration as the most promising NDI technique for composite bridges in terms of the coverage of damage area, global and local damage identification, assessment of severity of damage, and combination of health monitoring systems. Vibration-based, model-based methods provide global as well as local information on structural health conditions and do not require direct human accessibility to the structure. Such techniques are cost-effective and easy to operate, and have the potential for online damage detection with appropriate structural modeling. Structural health monitoring is an emerging technology. Successful development and implementation of the technology could lead to a reduction in the costs associated with maintenance, minimization of downtime to avoid economic loss and, most importantly, would help to ensure the safety of composite structures.

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ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
AE	Acoustic Emission
ASTM	American Society for Testing and Materials
AU	Acousto-Ultrasonic
BMS	Bridge Management Systems
CCM	Center for Composite Materials
CEE	Civil and Environmental Engineering
CDS	Characteristic Damage State
CFRP	Carbon Fiber Reinforced Polymer
DelDOT	Delaware Department of Transportation
DIA	Damage Index Algorithm
DMA	Dynamical Mechanical Analysis
DOT	Department of Transportation
DSC	Differential Scanning Calorimetry
DTI	Delaware Transportation Institute
ECT	Eddy Current Testing
FE	Finite Element
FHWA	Federal Highway Administration
FCM	Fracture Critical Member
FRP	Fiber Reinforced Polymer
GCLAD	Gas-Coupled Laser Acoustic Detection
GFRP	Glass Fiber Reinforced Polymer

GPR	Ground Penetrating Radar
GPS	Global Positioning System
IC	Integrated Circuit
IR	Infrared
IT	Information Technology
ISO	International Standard Organization
LPI	Liquid Penetrant Inspection
MEMS	Micro-Electronic and Mechanical System
MPI	Magnetic Particle Inspection
NBI	National Bridge Inventory
NDE	Nondestructive evaluation
NCHRP	National Cooperative Highway Research Program
NDI	Nondestructive inspection
NDT	Nondestructive technique
PE	Pulse Echo
QA	Quality Assurance
QC	Quality Control
RTM	Resin Transfer Molding
SCRIMP	Seemann Composite Resin Infusion Molding Process
SEM	Scanning Electronic Microscope
SHM	Structural Health Monitoring
SIR	Sub-surface Interface Radar
SWF	Stress-Wave Factor
TDR	Time Domain Reflectometry
UD	University of Delaware
UD-CCM	Center for Composite Materials, University of Delaware
USNA	United States Naval Academy
NSWCCD	Naval Surface Warfare Center, Carderock Divison
UT	Ultrasonic Testing
VARTM	Vacuum Assisted Resin Transfer Molding

APPENDIX A: Types and components of traditional bridges

A bridge is a structure, including supports, erected over a depression or obstruction such as a waterway, highway or railway, and having a track or passageway for carrying traffic or other moving loads. Historically, wooden and stone bridges were mainly built before the Industrial Revolution and steel arch and truss bridges followed. In the 20th century, designers of steel arch, suspension and cantilever bridges have prevailed and the new technologies of reinforced and prestressed concrete have allowed an unprecedented expansion of smaller-scale bridge-building worldwide (Brown, 1993).

A.1 Bridge types

Bridge types are categorized by their appearances, materials used and design requirements, as shown in Table A.1. A truss is essentially a triangulated assembly of straight members that has two major characteristics. The primary member forces are axial loads and the open-web system accommodates a greater depth than an equivalent solid-web girder. The cable-stayed bridge has become a strong competitor to the steel truss and made the latter less favored in modern design. Girder bridges have two types: plate girder and box girders. Advantages of a plate girder bridge include that they are of simple design and construction and use readily available materials. The box girder bridge is a good solution when bending and torsion are major concerns. The suspension bridge offers an elegant design when presented with spans of significant length over impressive physical obstacles. The steel arches provide for an attractive-looking structure while also eliminating the need for a pier in the river. Arches have been designed and built in a wide variety of bridge configurations of steel and concrete. Each of the bridge types requires NDI techniques.

Table A.1 Bridge types

	Names	Characteristics
Appearance	1. Truss 2. Arch 3. Reinforced concrete 4. Suspension bridge 5. Girder bridge	1. Axial force 2. No need for piers 3. Designable as a unit 4. Long span, elegant shape, complex 5. Popular and easy design
Material	1. Steel 2. Concrete 3. Timber 4. Masonry 5. FRP composite	1. High strength and high corrosion 2. Expensive, crack, spall, delamination 3. Decay, weak in moisture 4. Expensive, historic bridge 5. Strength/weight, Chemical inertness
Girder	1. I-beam or plate girder 2. Box beam girder	1. Simple, easy standardization 2. Bending and torsion endurable

A.2 Bridge components

A bridge consists of the superstructure and substructure. The superstructure comprises all the components of the bridge above the supports while the substructure consists of all elements required to support the superstructure and overpass roadway. Basic superstructure components consist of the following: wearing surface, deck, primary member and secondary member, whose function is to carry loads from the deck across the span and to the bridge supports. The deck's purpose is to provide a smooth and safe riding surface for the traffic utilizing the bridge. The primary members such as stingers or girders distribute loads longitudinally and are usually designed principally to resist flexure. Secondary members are bracing between primary members that are designed to resist cross-sectional deformation of the superstructure frame and help distribute part of the vertical load between stringers. Slabs are usually made of concrete while stringers or girders are made of steel or prestressed concrete. Basic substructure components consist of the following: abutment, piers, bearings, backwall, footing, and piles, whose function is to transfer the loads from the superstructure to the foundation soil or rock. Abutments are earth-retaining structures that support the superstructure and overpass roadway at the beginning and end of a bridge. Piers are structures that support the superstructure at intermediate points between the end supports (Tonias, 1995). The bearing provides an interface between the superstructure and the substructure.

A.3 Bridge materials

Timber: Timber bridges represent 8% of the total number of bridges in the United States. Many of these are very old, but timber bridges are gaining a resurgence in popularity (Hartle, et al., 1995). There are many characteristics that make wood a valuable material for use in bridges, including that it is a renewable resource, strong for its weight, relatively inexpensive, readily available, resistant to de-icing agents, resistant to damage from freezing and thawing, and able to sustain overloads for short periods of time.

Concrete: Concrete is a mixture of various ingredients such as cement, water, air, and aggregates. When these ingredients are mixed together in the proper proportions, they chemically react to form a strong, durable construction material. The strength of concrete has a typical compressive strength of 6000 psi; however, its tensile and shear strength is only about 10% of its compressive strength. Tensile steel reinforcement or prestressed concrete are generally used to supplement the limited tensile strength of concrete.

Steel: Steel is a widely used construction material for bridges due to its strength, relative ductility, and reliability. It also is versatile construction material since it is available as wire, cable, plates, bars, and rolled shapes. Accordingly, many of the nation's largest bridges are constructed primarily of steel.

Appendix B: Defects and inspection areas of traditional bridges

The purpose of a bridge inspection is to report the condition of a bridge and then assess the integrity, safety, and load-carrying capacity of the bridge. Therefore, the bridge inspector must gain an understanding of the various types of deterioration that can reduce the bridge's integrity, safety and capacity. It also is important to know the basic mechanics of bridge components. For example, truss members are designed for both compression and tension forces, while cables are designed for tension forces and columns for compressive axial forces. The following subsections define various defects related to the material properties and mechanics of the bridge components. First, some typical problems are highlighted that are encountered in bridge superstructures and substructure elements. Next, defects are addressed in view of materials. Attention should be carefully paid to inspect Fracture Critical Members (FCMs). FCMs are tension members or components whose failure will produce collapse of the structure. Fatigue is known to be the primary cause of failure in FCMs. The inspection of FCMs is an important and extremely complicated issue.

B.1 Defects and inspection areas by traditional bridge components

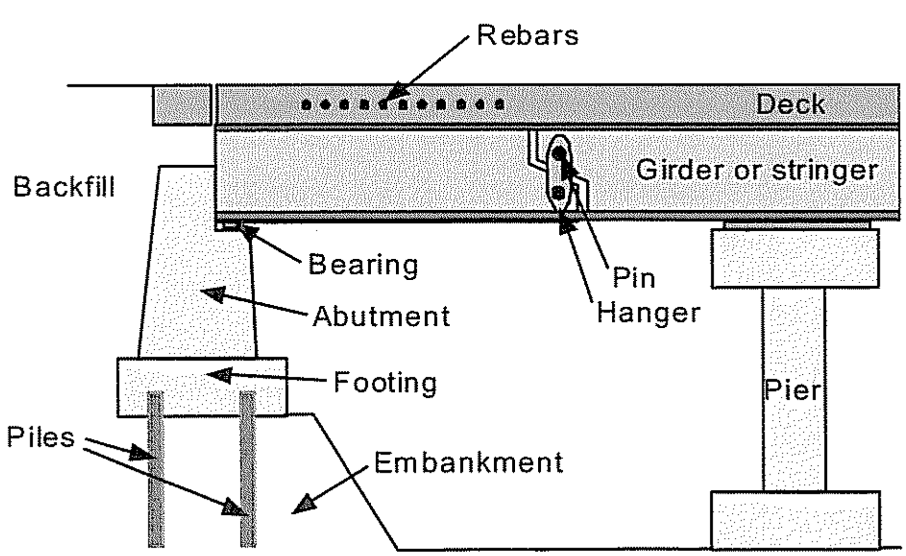


Figure B.1 Schematic diagram of concrete bridge and its representative components.

Figure B.1 shows the elevation view of the slab-on-stringer bridge, which is the backbone of the modern transportation network (Tonias, 1995). Visual inspection targets components such as deck and girder, pier and abutment, and steel components such as pin and hanger, girder or stringer, cables and truss.

Beam and girders: Beam and girders are designed to resist shear forces and bending moments. Steel beams and girders are main load-carrying members and should be examined for splitting, cracking, and excessive deformation or deflection. They also should be checked for cracking and corrosion, especially along the upper flange, around rivets and bolts, at contact surfaces, where moisture may enter between the flanges, at details at gusset and diaphragm connections, and at bearing assemblies. Box girders, unlike plate girders, must be inspected carefully on both the inside and outside of the boxes (lighting equipment is needed to inspect the inside of the box). FCMs are prone to crack in the tension zone, for example, at stiffeners, welds, and in areas such as the pin and link, the suspended span, and the cantilever span in a continuous structure. Irregular welds and the area around connections should be examined closely.

On the other hand, concrete T-beams should be checked for possible cracking, especially in the stem area, and for possible concrete deterioration, especially over bearings. Prestressed concrete girders should be checked for misalignment, cracking and concrete deterioration. All bearing devices should be examined to ascertain that they function properly.

Decks: Deterioration of concrete decks can occur in three basic forms: scaling, spalling, and cracking. Spalling is related to reinforced steel corrosion and is accelerated with the increased use of chlorides. Cracking occurs when tensile stresses exceed the tensile strength of concrete. Decks are typically replaced every 15 to 20 years, primarily due to deterioration caused by corrosion of the reinforcing steel. On the other hand, a main problem with steel decks relates to weld failure and cracks at connections. Deck joints are in a very corrosive environment so that rusting of exposed steel is a problem and steel plates exhibit the problems associated with steel. Scale should be removed by hammer when corrosion exists.

Substructures and waterways: A high percentage of bridges in the United States have no design plans available so that no information is available regarding the type, depth, geometry, or material incorporated in the foundations. These unknown bridge foundations pose a significant problem to the state department of transportation offices because the FHWA is requiring state DOTs to screen and evaluate all bridges to determine their susceptibility to scour. Bridge scour occurs during periods of high flow. The scour is the result of the force of the water removing bed materials from under the bridge. Foundation depth information is needed for performing an accurate scour evaluation at each bridge site, along with as much other information on foundation type, geometry, materials, and subsurface conditions as can be obtained (NDHRP, 1996). Footings such as piers and abutments should be checked for the potential of scour, undercutting, movements, and failure of materials. The inspection should ensure that the waterway is not obstructed, but carries the free flow of water. Piles in the splash zone and below the water surface should be checked for corrosion, deterioration, and section loss, with special attention given to exposed piles near saltwater. Waterway inspection requires one to check bank erosion, channel misalignment and hydraulic opening.

B.2 Defects and inspection areas by traditional bridge materials

Timber bridges: Timber highway bridges are relatively few and generally located on the secondary system. Timber is essentially anisotropic. Although wood is an excellent material for use in bridges, untreated wood is vulnerable to damage from fungi, parasites, and other sources such as fire, impact and collisions, abrasion and wear, weathering and warping, and overstress. Decay is the primary cause of timber bridge replacement. The major cause of decay is living fungi. Parasites tunnel in and hollow out the insides of timber members for food and shelter. Marine bores are water-borne and cause their most severe damage in the area between high and low tide water levels.

Concrete bridges: Among the common materials found in bridge, concrete is the most widely used. The main reason being that it is relatively inexpensive, readily available, and durable. Concrete, however, may deteriorate by cracking, spalling, scaling and corrosion of embedded reinforcement. Concrete superstructures may have the form of slab, box beam, T-beam, stringer, prestressed girder, or segmental construction. A common problem in these structures is that the wearing surface is an integral part of the system, and any loss of bond between the steel and the concrete because of spalling will accelerate corrosion and wear. This loss of bond is especially critical when the top reinforcing steel is part of the slab resistance to negative moments. Careful examinations are required especially for bearing spalling, diagonal cracks on the supports,and flexural cracks in tension area. Curbs and sidewalks should be examined for cracks, scaling, spalling, and deterioration. All handrails should be checked for possible damage from traffic impact.

Reinforced concrete structures are designed with the expectation that the concrete would crack in tension areas. Subsequently, finding cracks in a reinforced concrete structures is not alarming. It is, however, designed to be crack free in tensile areas. If cracks are found in tensile areas, they should be noted and reported. Only wide cracks may cause significant problems for the reinforced concrete but all cracks may initiate serious problems for the prestressed concrete.

Steel bridges: Flaws vary in size from very small undetectable nonmetallic inclusions to large inherent weld cracks, such as poorly made welds, rough edges resulting from shearing, punching, flame cuts or small holes. Such flaws may develop into cracks. As the number of loading cycles increases, the crack may grow into the materials until the rate of growth becomes unstable and fracture occurs. A problem common to most types of steel bridges is rust. Corrosion, fatigue crack, and damages by collision, overload and heat should be examined, especially for the shear and flexure zones, diaphragm, areas exposed to traffic, and fatigue-prone details. Loss of cross-sectional area of structural members or rust of bearings and hinge devices also are of major concern because of the obvious reduction of the structural capacity. The tension members of beam and girders are FCMs so that corrosion and cracks should be checked while buckling should be examined for the compression members. Beams and girders are particularly susceptible to cracking when joints and connections are welded, or when they include certain types of structural details such as partial-length cover plates, sharp re-entrant corners and cantilever brackets. Cracks perpendicular to primary stresses are very serious.

Likewise, trusses are especially susceptible to damage by rust and corrosion because of the numerous pockets created by the framing and the connections of the various members. The primary tension members of a truss are considered to be FCMs and deserve special attention. FCMs in a truss are connections such as eyebars, and pin and link, as well as the bottom chord in middle spans, the top chord over the pier and the appropriate diagonals. Most of the problems associated with truss bridges also are common in steel-arch structures. If deck and floor systems are supported by cable hangers, they must be watched closely for corrosion and wear at connection joints and where the cables enter a terminal socket. The key locations of FCMs are the floor beam connections, the hanger connections and the knuckle area.

The main suspension cables in suspension bridges are examined carefully to ensure that their protective covering is in good condition and provides protection of the steel against corrosion. Special attention should be given to the zones adjacent to the cable bands, at the saddles over the towers, and at the anchorage areas.

Steel parts extended into concrete should be examined closely for corrosion in movable bridges and counterweight elements must be checked to ensure that all parts are sound and secure and be examined carefully for wear, corrosion and lubrication level.

APPENDIX C: Types of major NDI techniques in civil engineering

C.1 Visual and optical inspection

Visual inspection is the oldest and still the most important form of nondestructive test, in which the unaided eye is used to inspect articles, observe symptoms and detect abnormalities. Methods of aiding human visual perception have been, and are continuing to be, developed. Visual inspection requires adequate lighting. Optical imagery such as optical holography or video enhancement and data display has been chief among these aids. A disadvantage is that the paints or coatings on the surface may need to be cleaned or removed, and have to be repainted after inspection (Bray et al. 1992).

C.2 Thermal inspection

Thermal measurements have wide use for process control, in-process inspection and monitoring of thermal condition. This type of visual inspection depends strongly on the area, thermal properties, temperature of the materials and the type of defect. The inspection process consists of three primary functions:

1. Heating or cooling of the test object
2. Sensing the resulting temperatures or thermal gradients
3. Correlating the measurements with pre-established standards

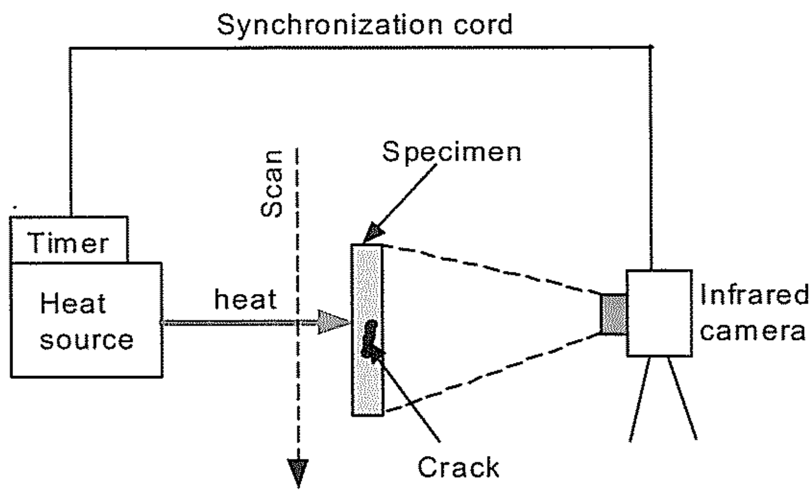


Figure C.1 Experimental setup of an infrared thermal inspection method.

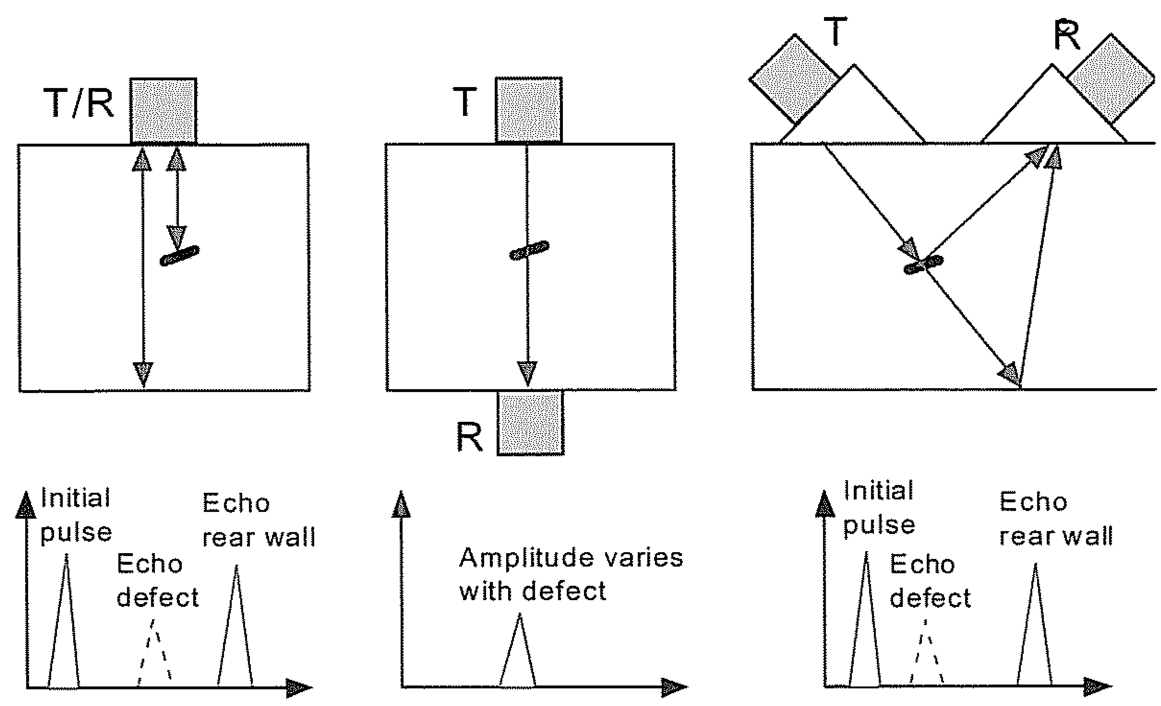
The thermal inspection methods include thermocouples, thermistors and temperature-sensitive paints and liquid crystals. Infrared (IR) thermal image method has several advantages: non-contact and remote sensing, entire area sensing, in-process sensing, and digital processing with color enhanced displays. It also has its disadvantages: emissivity change, coatings for low-emissivity surfaces, background radiation effects and high initial cost of equipment. In general, the technique is more sensitive to wear surface defects than to internal defects (Maldague, 1993). Figure C.1 shows an experimental arrangement of a thermal NDI method (Puttick, 1987). The radiant source, which is synchronized with an infrared camera via timer, is heating the back surface of a specimen, and a low-speed infrared camera is used to record the thermal images. A liquid crystal compound is applied by spraying a thin uniform layer to investigate the temperature variation on the specimen.

C.3 Ultrasonic testing

Imperfections and inclusions in solids cause sound waves to be scattered, resulting in echoes, reverberations and a general dampening of the sound wave. Sound waves of a frequency above 20 kHz also are known as ultrasonics. Frequency ranges of 0.1 to 15 MHz are normally used in NDI. In nondestructive testing, cracks, boundaries, or inclusions are detected by the change in impedance between the different media, when scattering and reflection of sound waves occur.

Defects perpendicular to the sound beam as small as $\lambda/4$ can be detected by ultrasonic testing (for example, a 1MHz signal can detect about 0.3 mm). The general dispositions of the various methods of ultrasonics and their oscilloscope displays are illustrated in Figure C.2. The arrangements can use one transducer only, as in pulse echo (PE) method (or A scan), which becomes the C scan method if the transducer moves along the surface of specimen. The transmission method and reflection method (or pitch-catch method) uses two transducers.

During Ultrasonic testing (UT), discontinuities in impedance are detected because they cause the ultrasonic waves to be reflected. A coupling liquid is used to provide a suitable sound path between the transducer and test surface. This is done to avoid a layer of air that has an impedance of 300 times scale than water between the probe and solid surface. The couplant must be as thin as possible; otherwise it may alter the direction of the ultrasonic beam. Various commercially made coupling agents also are available. Ultrasonics of high frequency can give rise to a good resolution with low penetration while ultrasonics of low frequency provides a high penetration but a low resolution. The large diameter transducers are chosen to produce a narrow focused beam that enhances the lateral resolution.



(a) Pulse echo method (A-scan) (b) Pulse transmission method (c) Reflection method

Figure C.2 Ultrasonic wave NDI testing arrangements.

UT equipment is capable of locating both surface and sub-surface defects in metals, including cracks, slag or other inclusions, segregation and lamination, pores or gas pockets, flaking, examination of welds, bonding inspection, shrink fit test, thin film, plate end, thickness measurements, porous solid, liquid level, air bubbles in pipe, etc. UT equipment is well-suited for analyzing possible defects in steel bridge elements and connection points. The equipment itself is relatively simple and portable; however, a skilled operator is required to ensure the equipment is properly calibrated and that signals are accurately interpreted (Blitz, 1996).

C.4 Acoustic emission testing

Acoustic emission (AE) is transient mechanical vibrations generated by the rapid release of energy from localized sources within materials. The AE method is a passive device. The sensor cannot detect a crack that is not growing. But, with adequate stimuli, AE method can be used to monitor behavior of materials, manufacturing processes and structural integrity, as well as many other specialized applications. AE can be used to monitor changing material conditions in real time and to determine the location of these emission centers as well. The advantages of AE are as follows:

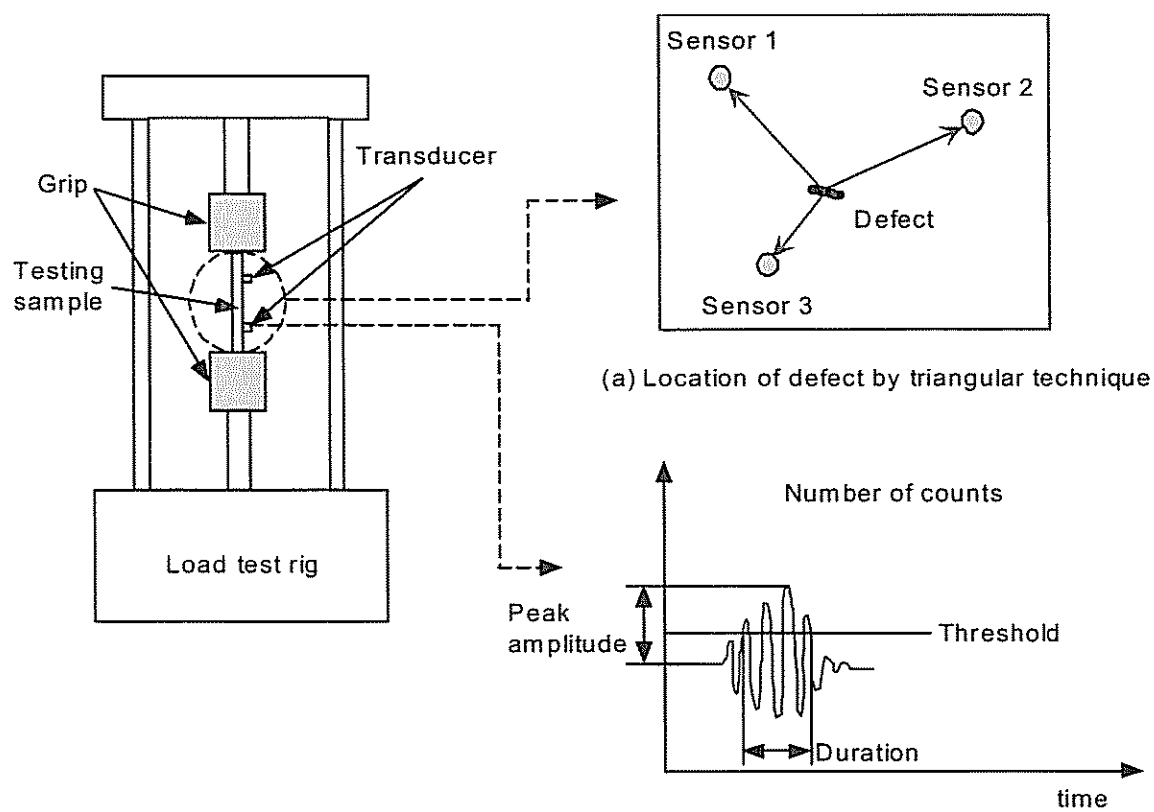
1. No equipment is required to excite a pulse
2. Highly sensitive to crack growth
3. The ability to monitor an entire system at the same time
4. Remote monitoring system.

The disadvantages of the technique are as follows:

1. Stabilized crack cannot be detected
2. The size of cracks or other defects cannot be detected.
3. The multiple numbers of travel paths from the source to the sensor in complex structures can make signal identification difficult.

Typical applications include on-board or on-site monitoring of aircrafts, pressure vessels, bridges, tanks welds, and so forth. Stimulated AE techniques also appear to be useful for monitoring certain types of composite materials. The most often used types of sensors include:

accelerometers, piezoelectric transducers, air-gap capacitive transducers, optical or laser sensors, and magnetostriction transducers. The most widely used method of quantifying AE signals is the ringdown counting technique, which measures the characteristics of the emitted signal as its amplitude decays. Figure C.3 shows the concept of laboratory AE test (Hill, et al., 1998). A defect can be located by the triangulation technique, which measures the sound elapsed time to arrive at each sensor as shown in Figure C.3 (a). The numbers of counts greater than threshold value, peak amplitude or rise time, etc. are used as parameters in characterizing AE events as shown in Figure C.3 (b).



(a) Location of defect by triangular technique

(b) Parameters to characterize emission events

Figure C.3 Concept of acoustic emission technique.

Since attenuation increases at higher frequencies, highly attenuating materials require more closely spaced sensors and more channels of instrumentation when the higher frequencies are monitored. Time differences in the arrivals of AE signals are used for locating the AE sources. To separate signals from ambient noise amplitude and frequency discrimination method, spatial or zone filtering, parametric filtering, and other types of filtering are used. Failure of loaded structures can be predicted by establishing AE characteristics for critical crack growth. A couplant such as wax or silicone grease is used to place transducers near areas of expected flaw growth or suspected areas of deterioration. AE techniques have been applied in a number of areas, including process monitoring and control, material studies, structural integrity assessments, mechanical equipment monitoring and leak testing. In most cases, other NDE techniques are used to verify the significance of a defect located by AE. In the case of FRP materials, peak amplitudes of AE indicate the type of event. Low amplitudes appear to be associated with matrix cracking and higher amplitudes indicate fiber breakage, which requires two threshold levels during the monitoring operation (Bray 1992).

AE is usually applied to what is called condition monitoring, the continuous testing of a structure or component while it is in service, rather than the more common practice of testing only at regular intervals. By placing a number of transducers in suitable positions, a source of emission can be located by the means of a somewhat complicated triangulation technique by determining the time differences between arrivals of a given emission at the transducers (Blitz, 1996).

C.5 Acousto-ultrasonics

Acousto-ultrasonic (AU) method, whose name is based on the fact that advantages of the acoustic emission and ultrasonic techniques are combined, utilizes two piezoelectric sensors attached to the same surface of an object being evaluated. As shown in Figure C.4, both sender and receiver are on the same materials surface with this method. AU method uses both time domain and frequency domain signals for the data processing. One of the transducers is excited using an ultrasonic pulser, which results in stress-wave propagation within the object, and the other transducer is used as a sensor for AE monitoring. This is applied mostly for evaluating the integrated effect of distributed damage or property variation within a component. When a conventional piezoelectric device is used as an exciter, some form of coupling media is needed to efficiently transfer the ultrasonic wave. If a pulsed-laser excitation is used as an exciter, concern may be directed at damage to the surface resulting from the localized high-energy density. The choice of detector frequencies is often governed more by the resonance of the object than by the frequency spectrum of the source of excitation. Once the detecting transducer has converted the surface displacements into a corresponding electrical signal, the quantified signal is then compared with a standard such as stress-wave factor (SWF). The signals can be quantitatively analyzed by using attenuation or velocity. The former is more sensitive to forms of damage that influence performance characterized by strength, while the latter is sensitive to damage-influencing performance characterized as stiffness.

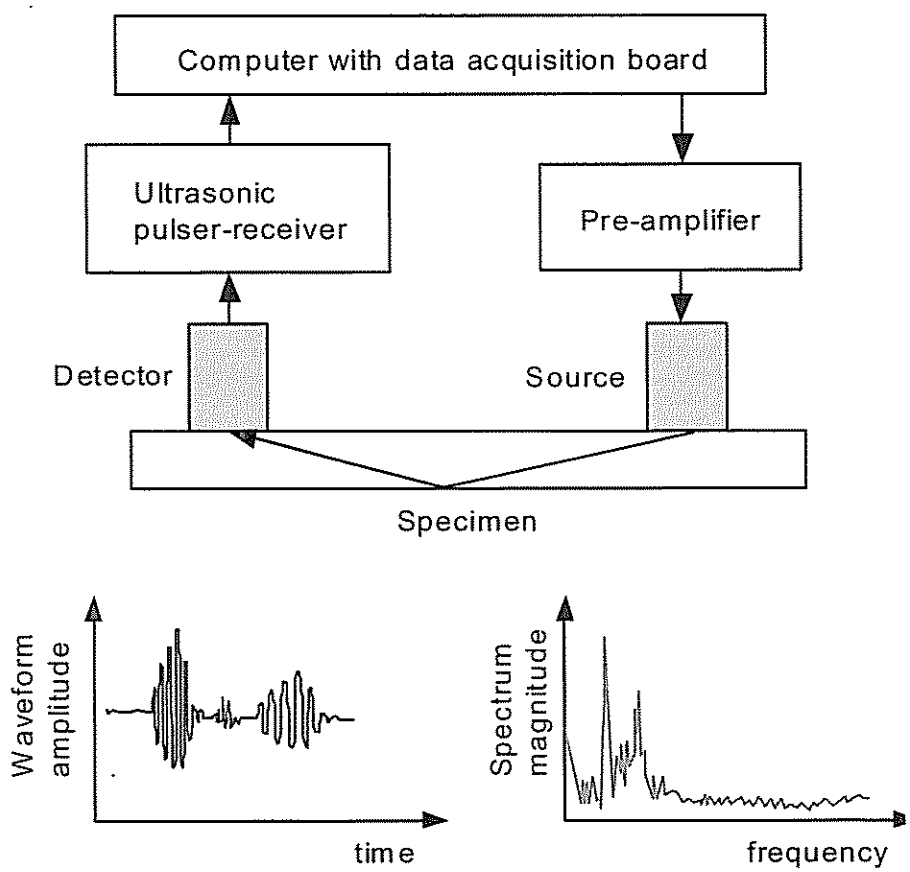


Figure C.4 Schematic of the acousto-ultrasonic configuration

C.6 Static loads testing

Bridges should be monitored during the passage of heavy vehicles to observe effects such as excessive vibrations and deflection. The 1994 AASHTO Manual for Condition Evaluation of Bridges recommends load testing as an effective means of evaluating the structural performance of a bridge and its components. This option applies particularly to bridges that cannot be accurately modeled by analysis or to structures whose structural response to live load is in question. A load test should be considered if it could provide a realistic assessment of load capacity. Prior to the test, a condition survey and structural analysis should be carried out with the intent to identify the critical components.

In general, static load testing involves static diagnostic test, impact test and proof test (Edberg, 1995). Static diagnostic tests employ loads, which are positioned at the point of interest and held stationary while responses are recorded; the results are then analyzed in a static model. The impact test is considered with static tests since they are used to determine the dynamic magnification factor for a specific bridge, which corresponds to a vehicle type. The test vehicles travel over the bridge at normal traffic speeds, which induces dynamic magnification of the static load but little actual dynamic response. In the proof test, instruments should be placed in the positions where the maximum expected strains would occur. Strain gauges are usually used to measure the amount of deformation in the structure, and this deformation is transmitted electrically to recording instruments (White, et al., 1992).

Finally, bridge rating is calculated with measured loads from the acquired data such as strain or deformation. A bridge load rating is used to determine the usable live load capacity of a bridge. Each member of a bridge has a unique load rating, and the bridge load rating represents the most critical one. Bridge load rating is generally expressed in units of tons, and it is computed based on the following basic formula:

$$\text{Bridge Load Capacity Rating} = \frac{\text{Allowable Load - Dead Load}}{\text{Rating Vehicle Live Load Plus Impact}} \times \text{Vehicle Weight}.$$

State DOTs utilize the AASHTO design code developed in computer programs specially designed for bridge rating. Rating a bridge is determining what gross vehicle weight can be safely carried by the bridge on a regular basis for the different vehicle types. Bridges that are seen at inspections to have significant deterioration must be evaluated and, if necessary, heavy vehicles must be limited. Bridge ratings are based on information in the bridge file, including the results of a recent field inspection. As part of every inspection cycle, bridge load ratings are reviewed and updated to reflect any relevant changes in condition or loading noted during inspection. Load ratings are routinely reported to the NBI for national bridge administration and also are used in local bridge management systems. Experimental load rating of bridges is an alternative to the current analytic rating methods utilized by state DOTs. Experimental load rating involves measuring the response of a bridge in the field to a controlled load. The information gained in the field test is then incorporated into the rating process to replace less accurate and generally conservative results obtained from conventional analysis. Edberg (1995) concludes that the best experimental load rating is static diagnostic testing, which can be easily expanded to include dynamic diagnostic testing. A new method is developed, which is primarily focused on assessing the safety of bridges for live loads and fatigue (Edberg, 1999).

C.7 Vibration testing

Advances made in microprocessors over the past two decades have made possible the relatively new technology of vibration analysis, which is quickly emerging from the specialized and developmental stage to become a popular and valuable industrial application (Pagliaro, et al., 1998). Flaws and variations in material properties may alter the vibrational characteristics of structures. Vibrational analysis for NDI purposes is called modal analysis, which can be acquired from experimental dynamic responses with excitation signals, and then spatial parameters are estimated inversely. A vibrational test involves the measurement of either the natural frequencies of the material or the rate of attenuation (Maia, et al., 1997).

Figure C.5 shows the schematic approach used to identify material properties as well as to locate damages of a structure. The measured modal parameters of the existing damaged structure and the theoretically generated modal parameters of the undamaged structure are used in combination with a proven damage detection method to generate possible locations and severity of damage in the existing damaged structure relative to the undamaged structure (Stubbs 1999).

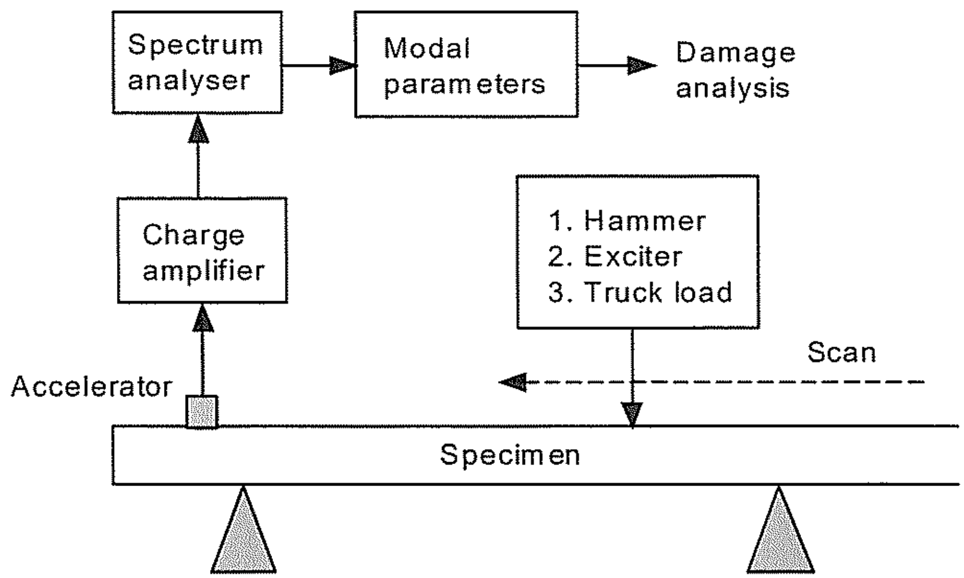


Figure C.5 Schematic diagram of vibrational NDI method.

There are two types of dynamic excitation methods. One is traditional -- the vibrational test by a hammer, shaker, or exciter, which is usually used in modal analysis of a structure. The other vibrational test is induced by traffic or wind. Dynamic responses of a bridge depend on the structural performance and can be expressed in terms of the dynamic parameters of the structure in which variation is related to its actual integrity. The advantage of the traffic-induced dynamic response of a bridge measurement is that the bridge must not be closed to traffic during the test (Casas, et al., 1990). Considerable anisotropy in the behavior of the structure means that small changes in the frequency of the mode shape and the level of damping may prove difficult to analyze. Likewise, temperature effects from one signature analysis to another may equally prove difficult to interpret (Martin, et al., 1998).

Due to a decrease in the cost of electronic equipment necessary for bridge testing, combined with the increase in the need to fully utilize bridges in the aging infrastructure, load rating has become a feasible option, which is regularly practiced by some DOTs. Over the past 30 years detecting damage in a structure from changes in global dynamic parameters has received considerable attention from the civil, aerospace and mechanical engineering communities (Farrar, 1998). Damage is usually detected by the examination of changes in modal parameters such as natural frequency, mode shape and structural damping obtained from the frequency response functions. Ongoing investigation include damage detection by using a modified

Laplacian operator without an undamaged base structure (Ratcliffe, 1997), by modal sensitivity parameters (Kashangaki, 1995), by a traffic induced vibration (Casas, et al., 1990; Miller, et al., 1992), and comparison of damage identification algorithms (Farrar,1998). Applications of vibration techniques include void evaluation under concrete slabs, offshore platforms, determination of structural degradation, etc. (Rens,et al.,1997). Vibration NDI method is more sensitive to microscopic damage than ultrasonic method and X-radiography (Boxwell, 1997). In addition, it enables economical planning and scheduling of maintenance and repair of bridge by monitoring (Maalej, et al., 1998) or with embedded smart sensors (Pines, et al., 1998). Global defect monitoring methods such as vibration tests, strain monitoring and AE can be used to accurately examine continuous damage changes in bridges with remote communications (Pines, et al., 1998). Direct-sequence, spread-spectrum, ultrasonic evaluation (DSSSUE) technique is used to globally evaluate large, complicated structural systems (Rens, et al., 1997). An advance vibration technique has been developed by Laser Doppler vibrometry, which has high resolution at a great distance so that the applications of large civil structures are possible (Castellini, et al., 1998). A system with multiplexed optical fiber sensors provides capability for distributed sensing of strains in large structures such as bridges (Ansari, et al., 1998). Vibration NDI techniques are widely used for field testing bridges (Farrar, et al., 1998). However, methods and procedures that encompass the overall response of a structure to an external stimulus can serve a broad scope and deserve further development. Recently, system identification techniques have been employed at the research level that obtain a structural model from the response data such as modal parameters, or time domain signals. Using ambient vibrations to perform system identification creates the possibility of continuously monitoring bridges, even from a remote location. The Federal Highway Administration (FHWA) sponsors research and development in global bridge monitoring, such as modal analysis, and embedded sensors and interferometry, that can detect and quantify significant changes in the complete structure (Chase, 1994). A previously reviewed advanced static load test and a dynamic monitoring method are complementary to each other so that a rating of the bridge is possible in the same manner as is employed with a static diagnostic test.

C.8 Radiography

Radiography is the technique of obtaining a shadow image of a solid using penetrating radiation such as X-rays or Gamma-rays. The contrast in a radiograph is due to different degrees of absorption of X-rays in the specimen and depends on variations in specimen thickness, different chemical constituents, or to scattering processes within the specimen. W. C. Roentgen discovered the X-ray in 1895 with his cathode-ray tube and Villard showed γ-rays in 1900 from uranium ores. X-rays are generated when an electron beam impinges on a solid target. X-rays can range in energy from less than 0.1 keV to more than 100 MeV. The lower energy X-rays are described as soft X-rays, known as Grenz-rays. Gamma-rays are electromagnetic radiation emitted by the disintegration of a radioactive isotope and have energy from about 100 keV to well over 1 MeV. Figure C.6 shows a schematic diagram of an X-ray NDI method. Oriented cracks are illustrated with their resulting effects on the intensity profile of the radiograph (Cartz, 1995). To obtain useful radiograph, the proper procedure is as follows:

1. Preliminary visual examination of the object,
2. Proper selection of energy levels,
3. Recording the image,
4. Interpretation of the radiograph.

High energy X-rays penetrate thicker specimens. Photographic films are used extensively for radiography. The differences between radiography and tomography are in that the detector of tomography measures the amount of radiation reflected or backscattered from the defects in the materials. In conventional radiography by transmission, the contrast of a defect is due to absorption differences along the total path that the X-ray transverses and there is no attempt to determine the distance along that path of the position of the defect. The position of the defect is revealed more precisely if Compton scattered X-rays are used.

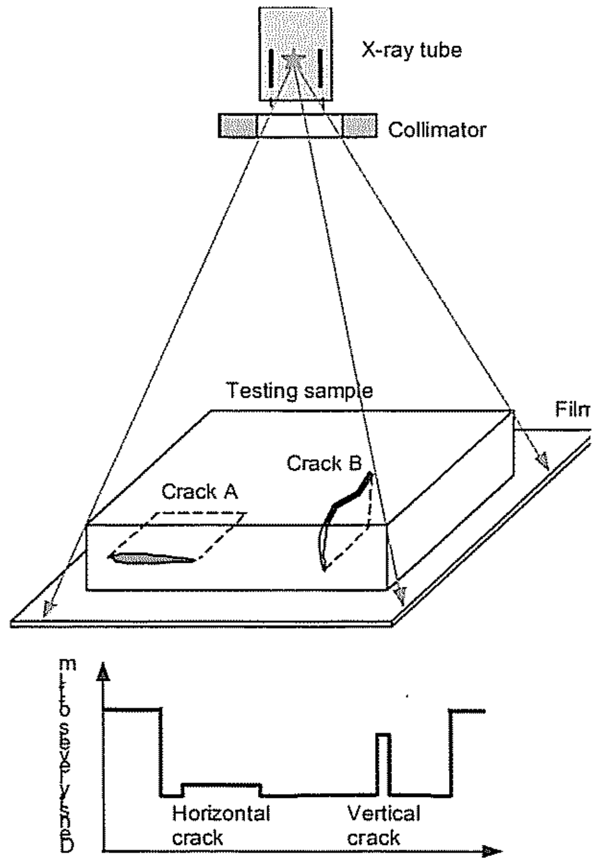


Figure C.6 Schematic of an X-radiographic system.

Both X-ray and Gamma-ray inspections are capable of locating surface as well as sub-surface defects such as cracks, incomplete weld fusion, slag and other inclusions, incomplete weld penetration and pores or wormholes. Both X-ray and Gamma-ray system have a common drawback. Both represent potential health hazards from radiation. In addition, both systems require well-qualified technicians for proper operation and personnel skilled in the interpretation of the information recorded on the films.

C.9 Liquid penetrant inspection

Liquid penetrant inspection (LPI), or Dye penetrant inspection, is a simple yet effective method of examining surface areas for cracks, defects, or discontinuities. Its origin almost certainly was in the observation by blacksmiths that quenching liquids could be seen to seep out of cracks and stain the surface after quenching a hot piece of ironware. LPI is relatively simple to carry out, and there are few limitations due to specimen material or geometry. In addition, it is inexpensive. Relatively little specialized training is required to perform the inspection, however experience is very helpful. The method is restricted to defects that are open to the surface. The steps in LPI are as follows:

1. Surface preparation
2. Penetrant application
3. Penetrant dwell
4. Excess penetrant removal
5. Developer application
6. Inspection
7. Clean surface.

LPI is used to detect defects such as welds, or defects in aircraft parts, locomotive parts, composite and bonded structures, or in light-alloy foundries (Cartz, 1995). The finest cracks that can be detected using LPI have been estimated to be about 5 μm wide, by 10 μm deep. However, LPI is not suitable for the location of subsurface defects. Commercial dye-penetrant kits provided in spray cans are readily available and are inexpensive.

C.10 Magnetic particle inspection

The magnetic lines of force are highly distorted across the crack, which is known as the magnetic leakage field. Figure C.7 shows a schematic diagram of magnetic particle inspection (MPI) (Blitz, 1991).

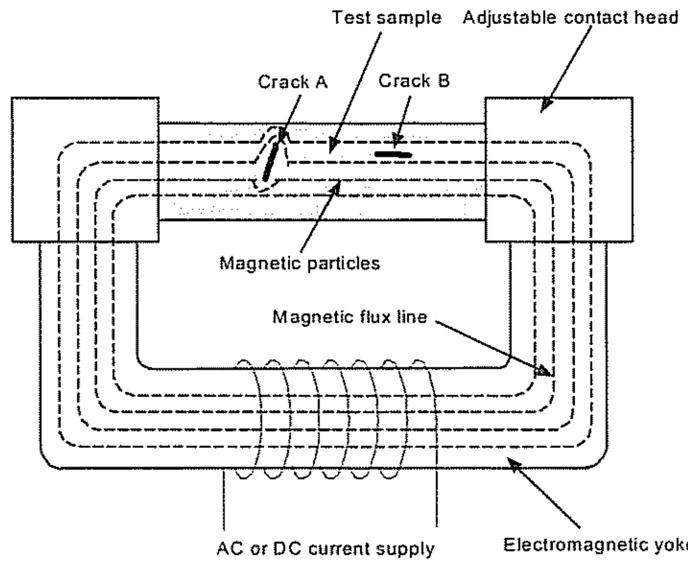


Figure C.7 Schematic diagram of magnetic particle inspection.

When the direction of magnetization is perpendicular to the flaw (eg, Crack A), the magnetic lines of force are distorted and the flaw can be detected. Crack B, located parallel to the magnetic field direction, does not result in any magnetic leakage field and is therefore not detectable with MPI. That is, a surface crack along the line between the prods may be detected readily, but a crack at right angles to the prod's axis may go undetected. Therefore, it is desirable to orient the prods in more than a single direction when checking for defects.

The procedure of MPI is that the workpiece surface must be cleaned, the specimen magnetized, magnetic particles spread over the surface, and the excess removed. MPI can be used to test some but not all ferromagnetic materials. For example, relative permeability is at least 300 for $H < 2500$ A/m. The magnetic powders have high permeability so that they become magnetized easily by small magnetic field. Crack visibility increases with crack lengths up to about 0.5 mm, above which it is essentially constant. MPI method can detect the sub-surface cracks up to a certain depth. Alternating currents produce a skin effect and are therefore useful for detecting surface defects. Full-wave rectified alternating current has a deeper penetration into the part and is used to detect sub-surface defects. Half-wave rectified alternating currents are advantageous for dry magnetic powder examination due to the pulsating nature of the electric and magnetic fields, which result in higher mobility of the dry powder. A magnetic field can be measured using a Hall-effect gauss meter. MPI can be used in all phases of the iron and steel industry such as solidification, rolling, forging, casting, welding and finishing.

C.11 Eddy current testing

A varying electric current flowing in a coil gives rise to a varying magnetic field. A nearby conductor resists the effect of the varying magnetic field and this manifests itself by eddy currents, which circulate in the specimen surface and are modified by the presence of discontinuities.

The detector instrument, such as an oscilloscope, will note changes in the time variation of voltage and current in the inspection coil in view of impedance. Figure C.8 shows schematic eddy current testing (ECT) of a specimen. The magnetic field from the inspection coil is opposed by an induced magnetic field from the eddy current. A surface crack modifies the eddy current and hence also the induced magnetic field. The signal captured by the detector is represented by a vector point on the oscilloscope screen. The various changes are recognized by experience and by calibration with known defects.

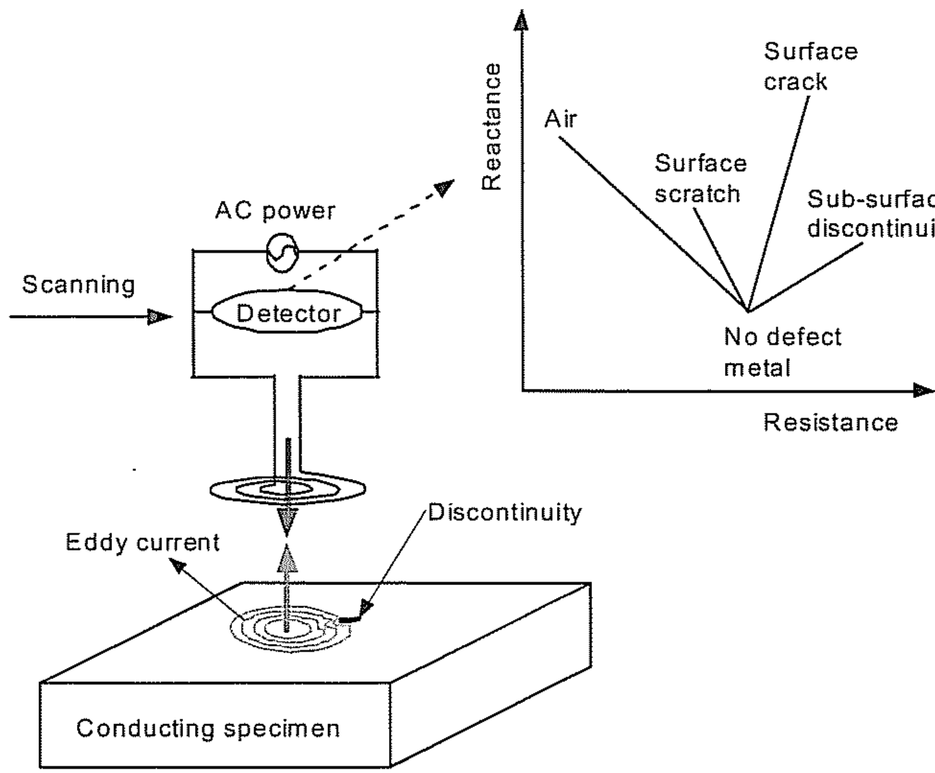


Figure C.8 Schematic diagram of eddy current inspection and impedance-plane diagram of the oscilloscope.

The higher the AC frequency, electrical conductivity, and magnetic permeability, the shallower the penetration, which is called the skin effect. Advantages of eddy current testing (ECT) are as follows:

1. Instantaneous results
2. Sensitive to a range of physical properties
3. Contact between inspection coil and specimen not required
3. Equipment small and self-contained
4. Can be miniaturized to observe discontinuities as small as 1 mm^3

Limitations are as follows:

1. Inspection of electrical conductors only
2. Depth of penetration restricted, probably only 1 cm from surface
3. Complication with ferromagnetic materials

4. Interpretation depends on skill of operator

5. Sensitive to many parameters

ECT is most convenient when all factors are fixed and controlled with only one factor at a time being allowed to vary when using the impedance method. ECTs are dedicated to particular applications such as crack test, electric conductivity test, thickness of coatings and surface conditions.

C.12 Ground penetrating radar

Ground penetrating radar (GPR) is also known as Sub-surface Interface Radar (SIR) due to its operating principles.

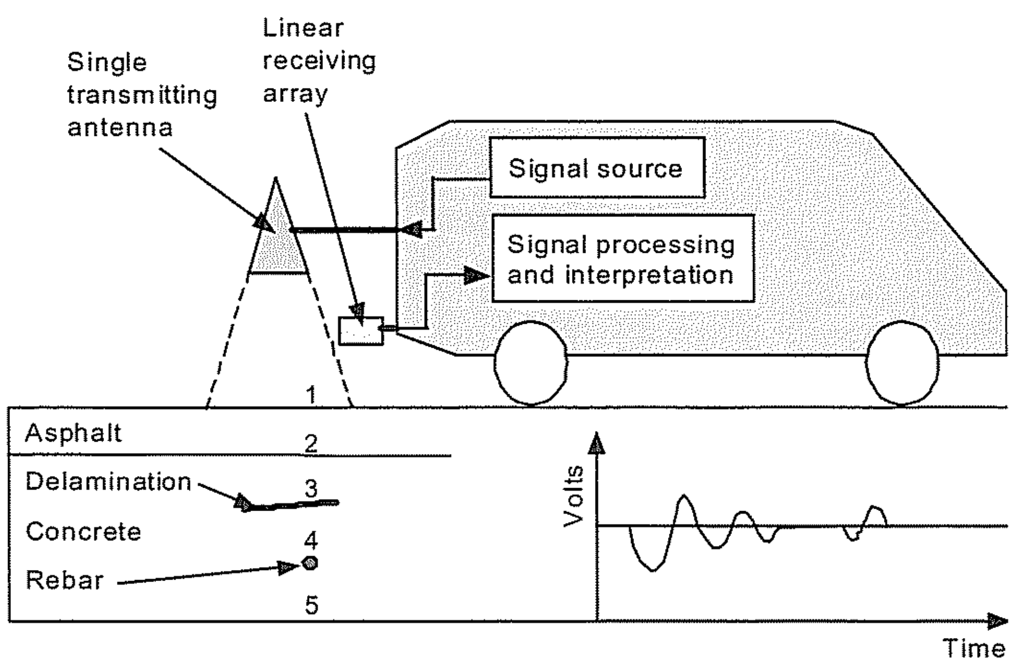


Figure C.9 Schematic diagram of ground radar inspection.

High-frequency electromagnetic pulses from a transmitting antenna are emitted and the time taken from any reflections from each pulse detected by a receiving antenna is measured. The antenna is pulled over the surface of the area being evaluated as the pulses are emitted. The propagation of the pulse through the soil is controlled by the dielectric constant of the soil, so that the pulse encounters an interface between materials with differing dielectric constants, and a portion of the pulse energy is reflected back toward the surface. GPR is used for testing soil-water contents. Figure C.9 shows a schematic diagram of a mobile ground-penetrating imaging radar system, which gathers data for high-resolution image reconstruction of embedded defects and features (Huston, 1999). A high-frequency antenna is necessary for good resolution, while a low-frequency antenna is needed to overcome clutter problems and for wave velocity calculation approximations (Martin, et al., 1998).

C.13 Time domain reflectometry

Time-domain reflectometry (TDR) is an electrical measurement technique that has been used since the 1940s to determine the spatial location and nature of various objects. It observes the echoes returning back from the device under test. A TDR is usually configured as shown in Figure C.10.

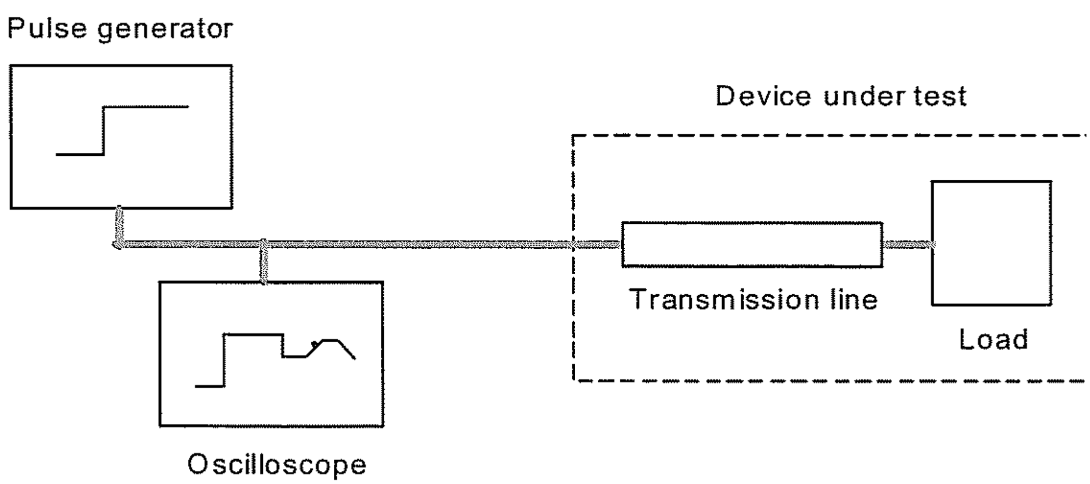


Figure C.10 functional block diagrams for a typical time domain reflectometry.

The pulse generator generates a fast rising step wave or pulse. This wave is launched into the transmission line. A high impedance oscilloscope is connected through a tee adapter to monitor the wave. An examination of the time delay and wave shape of the echoes allows us to determine the location and nature of discontinuities within the transmission line (Liu 1998). TDR is used for evaluation of bridge cables, detection of minutes, frazil ice, and so forth.

C.14 Structural health monitoring

Structural health monitoring (SHM) is an emerging technology that combines advanced sensing technology with a knowledge of material/structural damage characteristics to monitor the condition of structures in real time while in service. It has attracted significant attention in recent years for its potential wide applications, ranging from civil infrastructures, land/marine vehicles, and aircraft to aerospace transportation vehicles. SHM of civil infrastructure systems is especially cost-effective and necessary since these systems are generally the most expensive investments/assets in any country. In addition, these systems have a long service life compared with other commercial products, and are difficult to replace once they are erected (Chang, 1999).

The development of the technology involves multidisciplinary fields including sensors, signal processing, system integration and signal interpretation. Over the last decade, numerous types of new sensors have been developed. However, in real world civil structural measurements, conventional strain gauges and accelerometers are continuing to be used. Optical fiber-based sensors have been heralded as being the most promising for a wide range of civil infrastructure measurement. Piezoelectric sensors have a potential advantage in that they can act as both sensor and actuator. Using information technology (IT) and integrated circuit (IC) technologies, smart MEMS sensors can be designed to perform multiple functions such as built-in noise discrimination, self calibration and self-diagnostic capabilities.

The traditional NDI techniques tend to use direct measurements to determine the physical condition of the structures without history data. The accuracy of the diagnosis strongly depends upon the resolution of the measurements, which rely heavily on the equipment. However, the SHM techniques would use the change in the measurements at the same location at two different times to identify the condition of the structures. Hence, the history data is crucial for the technique. The accuracy of the identification depends strongly upon the sensitivity of sensors and the interpretation algorithm. Therefore, the NDI relies more on the equipment, but the SHM is more dependent upon the interpretation software.

C.15 Other advanced inspection techniques

There are many other NDI techniques that are not mentioned in the previous subsections, including acoustic wave sonic/ultrasonic velocity measurements, corrosion sensors, delamination detection machinery, electric methods, flat-jack testing, impact-echo testing, laser ultrasonic testing, magnetic field disturbance, magnetic field flux leakage, neutron probe for detection of chlorides, nuclear methods, pachometer, Pol-Tek, robotic inspection, and spectral analysis (Hartle, et al., 1995). Detailed explanations are omitted because these techniques have similar basic principles with the traditional techniques.

It also is helpful to understand destructive techniques, such as boring or drilling, Brinell hardness test, carbonation, Charpy impact test, chemical analysis, concrete permeability, concrete strength, endoscopes, moisture content, probing, reinforcing steel strength, Shigometer, and tensile strength test. These destructive techniques may be used for QA/QC purposes of selected samples or the supplements of NDI techniques. See *Bridge Training Manuals* for detailed explanations (Hartle, et al., 1995).

APPENDIX D: Review of NDI applications for civil infrastructures

About 40 percent of the bridges in the U.S. national inventory are structurally deficient or functionally obsolete (BIRL, 1999). The BMS system assesses defects and the physical condition of bridges by using data generated from quantitative bridge inspection methods such as NDI techniques. The following sections review various NDI applications of civil infrastructures focusing mainly on bridge applications. Destructive techniques also are included for comparison. They are used as a supplement to visual inspection and other more common inspection techniques.

D.1 Timber bridges

A visual examination can detect most damage types in timber bridge. Destructive tests include probing, boring, drilling, measuring moisture content, core sampling and use of the Shigometer. The Shigometer measures electrical resistance to detect rot in timber members. A sounding hammer or similar device is usually used to detect decay and occasionally the same investigation is processed with an increment borer or a nail and ice pick. Pol-Tek is a sonic testing device that is used to detect rot or other low density regions in timber poles (Hartle, et al., 1995). Ultrasonic testing can be used to detect cracks, internal flaws, discontinuities, and surface damage. Energy-decay techniques based on either impact or ultrasonic excitations appear to be most promising. Rapid NDI methods are needed for determining the presence of voids and differences in moisture content and density (Xanthakos, 1996), and further research is needed to develop easy and accurate methods of assessing timber deterioration (Rens, et al., 1998).

D.2 Concrete components of bridges

As a usual practice, most states use cores, soundings, physical dimensions, visual assessments and engineering judgment to arrive at design stresses and usable cross-sectional area. The widespread use of core sampling in determining the properties of concrete reflects the fact that samples can be easily obtained and strength test methods are reliable and relatively inexpensive. The method destructively measures several concrete properties such as carbonation, permeability, strength and moisture content. The disadvantages of the method are substantial damage after testing and that many samples are needed because of the non-homogeneous nature of concrete. A trend to avoid destructive testing is evident (Rens, et al. 1997, Xanthakos, 1996).

Techniques for detecting defects range from visual inspection to ground penetrating radar. Many states use visual examination routinely. All kinds of cracks should be examined with an optical crack gauge. Some states use polarization, liquid penetrant and pulse echo ultrasound techniques. Cracks can be evaluated by ultrasonics (Rens, et al., 1997, Beaudry, 1995). But the limitation of the ultrasonic test is being able to identify defects smaller than $\lambda/2$(for example, 1MHz signal can detect about 0.6 mm), and attenuation of the relatively high signal frequency due to dispersion in the concrete (Martin, et al., 1998). Coupling problems can be avoided by the air-coupled ultrasonic method (Strycek, et al., 1999). The ability to estimate the rate of corrosion in steel bars is essential in order to predict the remaining service life. Corrosion in steel reinforcing bars can be detected by an electro corrosion tool (Beaudry, 1995). Copper sulfate electrode tests can be used to measure the concrete's ability to conduct electricity. Less electrical resistance indicates a high chloride content and estimate of corrosion activity on the reinforcing bars. Rapid, remote-sensing methods based on radar and infrared tomography have been found to be very effective for detecting hidden deterioration in concrete (Xanthakos, 1996). Electromagnetic cover meters are widely used to locate the position and depth of embedded reinforcing steel in concrete, while radar has recently attracted considerable interest for its ability to locate and comparatively assess construction features including poor compaction voids, reinforcing bars, and cracking (Bungey, 1994). An ultrasonic technique was developed for locating major faults, but it has not been proven capable of detecting minor corrosion under shallow concrete covers in embedded prestressing steel components (NCHRP, 1992) while laser ultrasonic testing provides information about the quality of the concrete at various depths from the surface (Hartle, et al., 1995).

A full evaluation of concrete decks can be accomplished with sonic/ultrasonic acoustic wave velocity measurements. This method delineates areas of internal cracking (including delaminations) and deteriorated concrete (including elastic modulus values). Surface coating and sub-surface delaminations are effectively detected by an infrared thermography with visual imaging method (Washer, 1998; Thomas, et al., 1993). A neutron probe can be used to detect chlorides in construction materials by measuring gamma rays signals. Nuclear methods are used to measure the moisture content in concrete by neutron absorption and scattering techniques.

Ground-penetrating radar (GPR) is designed to image an entire lane width of a bridge deck in a single pass at highway speeds (Washer, 1998). GPR is a time-domain technique whose time is dependent on the dielectric constant of materials (Martin, et al., 1998). GPR also is used for concrete bridge health monitoring applications (Huston, et al., 1999). The surface size, shape, location, and extent of discontinuities can be determined by dye penetrant method (Beaudry, 1995). Damage from freezing and thawing are inspected with ultrasonic waves signals (Akhras, 1998). Air void and rebar images are constructed by pulse-echo microwave imaging (Lockwood, et al., 1997). Stress wave method has been used for a concrete NDI (Popovics, et al., 1994). Rebound and penetration methods measure the hardness of concrete and can be used to predict the strength of concrete (Hartle, et al., 1995). Acoustic emission (AE) receives and analyzes sound waves produced by events such as cracking or active corrosion (Beaudry, 1995). Reinforcing and prestressed concrete are not FCMs but a thorough inspection of nonredundant concrete members in tension should be conducted (Hartle, et al., 1995). In the 1996 survey, the main concern of state DOTs are bridge piers, decks, column caps and abutments, and the need for further research to evaluate in-plane concrete strength, deterioration, crack detection and growth (Rens, et al., 1998).

D.3 Steel components of bridges

Destructive tests may be necessary to determine the strength or other properties of existing steel on bridges for which the steel type is unknown. A specimen is removed from the bridges and used for the destructive tests such as chemical analysis, Brinell hardness test, Charpy impact test and tensile strength test.

Visual inspection for corrosion and section loss of fatigue cracks is most common. Paint should be removed to inspect the target surface. The most common test is the measurement of member thickness to estimate section loss caused by corrosion. Several companies are currently developing and marketing a system that uses high resolution video cameras on robotic arms attached to permanent falsework underneath the bridge. By remote telescanning, details can be visually monitored (Hartle, et al., 1995). Ultrasonics and penetrating dyes are the next common method after visual examination. Some states also use NDI such as X-ray radiography, MPI and eddy current methods (Xanthakos, 1996). A visual observation of structure and member alignment is the first step in a truss inspection. A crack in a weld in a diaphragm connection is hard to see. Therefore, to make it more visible, dye penetrant method can be used (FHWA, 1988). Ultrasonics is used for the inspection of pin and hanger (Thomas, et al., 1993; Washer, 1998), eye-bar, welds, bolts, internal flaw, and corrosion thickness. Impressive advances have been noted in the application of ultrasonic methods to the detection and identification of flaws in steel components. Further development will improve characterization and permit automated data processing. Corrosion levels of steel structures can be analyzed by infrared imaging technique with portable, large-scan ability (Rens, et al., 1997). Welds, connections, fatigue cracks, and bolt failures are inspected by AE and strain gauge monitors (BIRL, 1999), which can be used before and after retrofit solution to assess the effectiveness of the retrofit before a significant amount of money is spent on the rest of the structure. X-radiography is used for inspection of structural metals fabrication and weld qualification. It can be combined with computer tomography to enhance its performance. Critical welds are radiographed in the shop, where defects can be repaired. Magnetic method such as magnetic particle inspection (MPI) and eddy current (EC) inspection can be used to determine surface and sub-surface flaws, and stresses for ferromagnetic materials such as in steel bridges. MPI is used to aid visual inspection. MPI technique spreads fine ferromagnetic particles over a magnetized area so that surface and sub-surface discontinuities in ferromagnetic materials such as steel can be located (Beaudry, 1995). Computed tomography and Time Domain Reflectometry (TDR) are used for evaluation of bridge cable and cable anchors (Liu, 1998; Thomas, et al., 1993).

The FHWA has sponsored research related to steel bridges and has developed an eddy current technique, an ultrasonic imaging system for detecting cracks, and an acoustic emission (AE) weld monitor which successfully detected flaws as an in-process inspection tool during steel bridge fabrication, and a magnetic field device for cables and cable stays. An AE technique has been developed to monitor growth of fatigue cracks (Chase, 1994). Beams, connections, and pins as problematic components are main concerns of state DOTs. Further research for simplifying ultrasound, pin and hanger assemblies detection, and bolt tension by an AE techniques is required (Rens, et al., 1998).

D.4 Subsurface bridge foundation and under-water inspection

Surface tests such as sonic echo/impulse response, bending wave, ultraseismic, spectral analysis of surface waves and dynamic foundation response tests are used to determine the depths of an unknown bridge foundation. The major limitation of the surface methods, however, is that none of them can detect the presence of piles underlying a buried pile cap or a more massive abutment or pier wall. In the case of buried pile foundations, the parallel seismic test was found to have the broadest application for varying substructure and geological conditions, but it requires drilling boreholes (NCHRP, 1996).

The methods currently in use are the same as those of concrete and steel components tests, which are usually based on technology developed by the marine industry (Xanthakos, 1996). Piers and abutments can be monitored by TDR and pile tip can be measured with stress wave based on beam vibration theory (BIRL, 1999). The normal procedure for evaluating bridge scour is to inspect the river bed using probing rods, scour tracker, and sonar scour monitor, which can be replaced by a more advanced technique such as GPR (Martin, et al., 1998). Scour holes in river beds around bridge piers are detected by radar and sonar and by measurements of density, and moisture contents are measured by microwave absorption techniques (Bungey, 1994). GPR is used for pavement profiling, and the layer thickness and permittivity of asphalt and concrete can be estimated by using an inverse scattering approach (Spagnolini, et al., 1999). GPR has been used as an effective tool for monitoring changes in bedrock elevations at bridge footings (Beaudry, 1995). Further research is needed into remote scour monitoring and foundation examination (Rens, et al., 1998).

D.5 Pavements

The material properties of the pavement layers may be obtained by destructive testing such as drilled cores and sawed beams from existing pavements. However, destructive tests are expensive, time-consuming and have limited validity. The most commonly used NDI methods are: static tests, steady-state dynamic tests, impact load response tests, electromagnetic tests, and wave propagation tests (Popovics, et al., 1998).

A nuclear density testing device measures field density of pavement, nuclear asphalt contents in granular base and cohesive subgrade materials. A dynamic cone penetrometer measures the in-situ strengths of subgrade and base materials in an inexpensive and timely manner, but it is not a true NDI device because it requires access holes in the material. Profilers such as South Dakota Profiler, California Profilograph and GM Profilometer are used to measure a pavement roughness profile or to determine the ride quality of new concrete pavements. GPR could be used for determining marsh depths, locating voids in rock, locating underground tanks and assessing pavement thickness (Beaudry, 1995). Surface scanning radar is used for condition surveys of highway pavements (Bungey, 1994). NDI with a capacitor probe, which is related to dielectric properties of concrete, is developed to detect sub-surface deterioration of concrete (Diefenderfer, et al., 1998). TDR is used for detection of mines (BIRL, 1999) and for electromagnetic-based frazil ice detection system (Yankielun, et al., 1999). Significant progress has been made and the newer methods are more reliable, quicker and more practical than earlier techniques, but these new techniques still need to be perfected (Popovics, et al., 1998).

D.6 Dam and related structures

Many NDI techniques are used for a variety of waterways and dams. TDR is used for remote monitoring tools for mines (BIRL, 1999), frazil ice (Yankielun, et al., 1999). Seismic refraction measures the time that the seismic wave takes to reach one or more geophones, which measures the velocity of the material and indirectly yields the depth of the layer changes (Beaudry, 1995).

Appendix E: Vibration testing of Bridge 1-351 (June, 2000)

SUMMARY

Composite materials are gaining increased use as a structural material for large scale composite structures such as bridges. With structures of this size, methodologies for ensuring the structural integrity become very important. Although inspection techniques are available which can be readily implemented in a laboratory setting, few exist which can be used in the type of service environment to which the aforementioned applications are subjected.

This paper presents the results of an ongoing investigation into using broadband vibration data to monitor the structural integrity and health of an all-composite road bridge. More specifically, this paper presents results of the second inspection of Bridge 1-351 on Business Route 896 located in Glasgow, Delaware. The bridge consists of two E-Glass/vinyl ester sections (each 13-ft x 33 ft) joined by a longitudinal joint in the traffic direction. The section is a sandwich construction consisting of a 28-inch deep core and 0.5-0.6-inch thick facesheets. This bridge deck configuration is representative of the deck sections that have been manufactured for both the ship decks and causeways. This second vibration inspection was performed to determine if there was any degradation to the structure during its first year of service. Demonstrating the ability to determine the structural health and any degradation in properties of the Route 896 bridge in its service environment would illustrate the utility of the broadband vibration technique for Navy structures.

Vibration data were obtained from a mesh of 1050 test points covering the upper and lower facesheets of the bridge. The same mesh was used for both of the inspections preformed to date. From the modal information and the visualization of the data, several aspects of the structural behavior of the bridge will be reported and compared to last year's inspection. These characteristics include the quantification of the interactions between the bridge and abutments; the effectiveness of the longitudinal joint to couple the deck sections; the effectiveness of the core to couple the face sheets and the integrity and dynamic consistency of the entire structure. In addition, mode shapes and natural frequencies were identified and will be correlated with design calculations and vibration analyses conducted for this bridge.

The vibration data also were obtained as part of an effort to further develop a novel algorithm that uses these data to identify local perturbations of the state of the material (e.g. manufacturing defects, material degradation or service damage). This algorithm will detect and

spatially locate the features that may be present in the bridge. This technique has been successfully validated for locating damage in 1-D beam structures. The extension of this technique to the 3-D sandwich structure is currently in progress and will be discussed although specifics of quality assurance/quality control and health monitoring for composite bridge structures are still in their preliminary stages and will therefore not be discussed in detail.

The vibration data were obtained as part of an effort to develop a vibration-based non-destructive evaluation method suitable for long-term inspection of composite deck bridges. The report covers the data collection phase of the project and the results of a preliminary modal analysis.

CONTENTS

BACKGROUND

A research and educational collaboration initiated in the early 1990s by the University of Delaware Center for Composite Materials (CCM), the University of Delaware Department of Civil and Environmental Engineering (CEE) and the Delaware Transportation Institute (DTI) culminated in the installation of an all-composite bridge deck on Business Route 896 in Glasgow, Delaware. Other partners in the project were the Delaware Department of Transportation (DelDOT), the Federal Highway Administration (FHWA), and local industry and contractors including Hardcore Composites, Anholt Technologies, and James Julian, Inc.

The bridge is identified in the Delaware highway system as Bridge 1-351. It is situated at approximately 39°36.63'N 75°44.77'W and carries Route 896 over a small stream known locally as Muddy Run. During the vibration trial reported here, the Muddy Run water under the bridge was a few inches deep at the sides, increasing to about 3 feet in the middle. It was about 3-4 inches deeper than during the previous trial conducted in May, 1999.

Many details of the vibration analysis of this bridge are included in Reference 1 (May, 1999 report). The reader is directed to that reference for more details on the initial vibration-monitoring test and structural features.

The new bridge was installed and opened to traffic on November 20, 1998. This bridge carries a single-lane one-way road. Figure E.1 is a panorama of the bridge, compiled from several still images. In this picture, north is left and south is right.

Figure E.1: Panoramic View of Composite Bridge Deck

The focus of the vibration trial reported here was to obtain a large quantity of frequency-based vibration data from the bridge deck. These data are being archived for later work. This report includes the results of the modal analysis and a comparison with the baseline modal results from May, 1999. The results of applying the broadband damage detection method also are presented and discussed.

KEY PERSONNEL

The following were the key on-site personnel involved in the June, 2000 vibration trial:

United States Naval Academy (USNA) - Professor Colin P. Ratcliffe

Naval Surface Warfare Center, Carderock Division (NSWCCD) - Dr. Roger M. Crane

The Center for Composite Materials (CCM) - Professor John W. Gillespie Jr., Dr. Dirk Heider, Dr. Myung Keun Yoon

Delaware Department of Transportation (DelDOT) - A team of employees who closed the bridge and provided on-site support such as supplying a boat and ensuring that traffic did not pass the barricades.

SCHEDULE

Data capture was scheduled for the week commencing June 5, 2000. The following summarizes the actual trial phase of the project.

Monday 5th

0745	Initial discussions and final planning at CCM
0845	On-site. Weather was cool (high was high 60s) and dry. Checked grid marks remaining from the May, 1999 trial and measured and repainted those that were missing (worn off). Accelerometers were installed at same places as last year.
1110	Started data capture for impacting the top surface
1415	Completed data capture for the top surface. Weatherized underside accelerometers in preparation for overnight rain. Removed topside cables and accelerometers.
1445	Bridge reopened to traffic.
1445-1615	Traveled to "home base" in Maryland. Translated data and conducted provisional modal analysis en route.

Tuesday 6th

0745	Arrived on site. Weather was windy, cool (low 60s) and raining hard. Started laying out cables.
0800	Bridge closed to traffic. Re-installed topside accelerometers and ran cables. Waterproofed all connections and transducers. Prepared analyzer.
0826	Commenced data capture for impacting the underside of the bridge.
1119	Underside data capture completed. 512 data files were recorded. This represents 2.96 measurements per minute.
1126	Commenced data capture for topside. By now, it had stopped raining, but remained cool and windy. The top surface data capture was repeated in order to compare the modal results with the previous day's work and demonstrate repeatability.
1400	Topside data capture completed. 573 data files were recorded for this phase. This represents 3.72 measurements per minute. This is a higher rate than for the under side, because hitting the top was easier.
1425	Bridge reopened to traffic.
1430-1600	Traveled to "home base" in Maryland and translated data en route. The extra data set size precluded a modal analysis in this time frame.

BRIDGE GENERAL DESCRIPTION

A more detailed description, including a survey of the bridge dimensions, is included in Reference 1. For this project, the bridge can be considered to have three main components: two guardrails and one deck. Dynamically, these components are decoupled by rubber seals installed between each guardrail and the deck. The main components of the bridge are shown in the photographs of Figs. E.2-4, which are taken from Reference 1. The deck is the main component of interest for this project. It is approximately 26 feet wide and 32 feet long, and sits on abutments at the north and south banks. The deck was manufactured in two parts, each approximately 32 feet long and 13 feet wide. The two parts are joined by a longitudinal north-south joint, which can be seen in Figure 4, which shows the underside of the bridge.

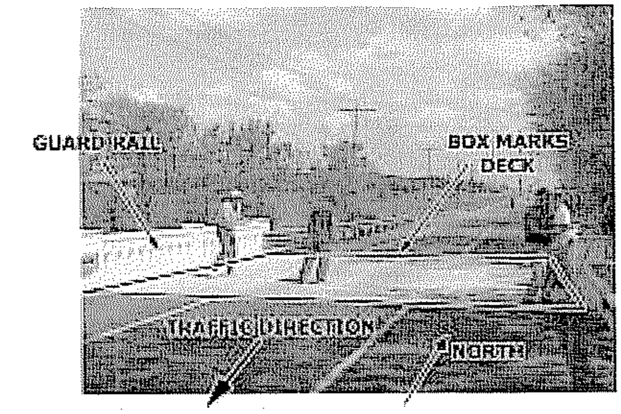


Figure E.2: View of Bridge Deck Components (Top)

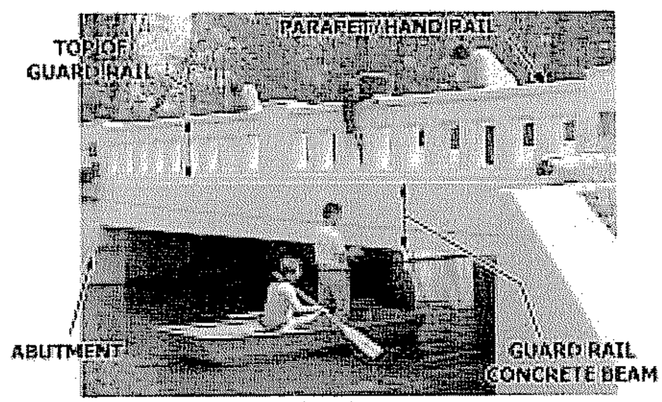


Figure E.3: View of Bridge Deck Components (Side)

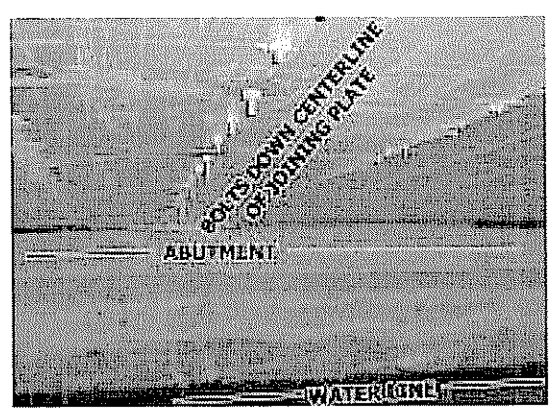


Figure E.4: View of Underside of Composite Bridge Deck, Looking North with Centerline Joint

SURVEY

As part of the May, 1999 trial, the bridge was surveyed. The global origin for all measurements was taken as the extreme southeast corner of the concrete guardrail at deck level; this being a point on the bridge thought unlikely to be damaged or moved by traffic or other accidents. The X-axis was parallel to the line of the bridge (pointing approximately north); the Y-axis was across the bridge (pointing approximately west); and the Z-axis pointed upwards. The axis origin was at the height of the top bridge deck. The small curvature of the deck surface was ignored; all test points on this surface being assigned zero Z-coordinate values. As a result of this origin location and ignoring the surface slope, all test points on the bottom surface had the same negative Z-coordinate value (-30 inches).

THE TEST GRID – GENERAL

The mesh of test grid points consists of two sub meshes: one on the top surface and one on the bottom surface. Both meshes were uniform, with an 18-inch spacing in the X- (north-south) direction, and a twelve-inch spacing in the Y- (east-west) direction. The grid point numbering was essentially south-to-north, starting at the east-most line and working westwards. Grid point numbers 1 through 4 were reserved for accelerometer positions (described later), so the first southeast grid point on the top deck surface was # 5. This point was located at global coordinate position (19.5", 21.0", 0"), this being about 6 inches onto the surface of the deck in both the X- and Y- directions. The highest-numbered grid point on the top mesh was # 550, at global position (379.5", 321.0", 0").

The grid points for the lower-surface mesh were directly under points on the top surface. For convenience, grid point numbers on the lower surface were set as the coincident upper surface number plus 600. Thus, for example, upper surface grid point # 271 has global coordinates (271.5", 165.0", 0") and lower surface grid point # 871 (= 271 + 600) has global coordinates (271.5", 165.0", -30.0").

The lower surface mesh has one less line of grid points at both the north and south ends. This is because the deck (and hence the upper surface mesh) extends over the supporting abutments, and it was not possible to test at these positions on the lower surface. Therefore, at the south end, grid points # 605, 626, … do not exist. Similarly at the north end, grid points # 625, 646, …do not exist.

During the May, 1999 trial, the grid points were marked with paint spots. For this trail, most of the grid points were still identifiable. The global origin on the top surface was easily identified. The original spots were refreshed with new paint. Most of the spots remained on what would be the shoulder of the bridge. The spots that were missing were in area where they could be worn off by traffic. The location of the missing spots was interpolated from adjacent existing identifiable spots.

The test grid was checked for accuracy when compared to a uniform mesh. The largest error was 1-inch, although this was primarily a bias error. Therefore, for consistency, where the spots were not in exactly the correct global positions, the old positions were reutilized.

THE TEST GRID – ACCELEROMETERS

Four accelerometers were used to record the motion of the deck. Two were located on the top surface and two on the bottom surface. The top surface accelerometers were placed at the center of a rectangle formed by four adjacent test mesh points. In May, 1999, the location of the bottom surface accelerometers was determined less precisely. Also, Reference 1 has an error in that Accelerometer D was identified at the wrong place. Based on detailed measurements taken in June, 2000, the accelerometers were at the following positions:

Upper surface Accelerometer “A”:

Centered in the mesh defined by points # 333, 334, 354, and 355

Global position: (262.5”, 207.0”, 0.0”)

Upper surface Accelerometer “B”:

Centered in the mesh defined by points # 96, 97, 117, and 118.

Global position: (154.5”, 75.0”, 0.0”)

Lower surface Accelerometer “C”:

Bounded by points # 927, 928, 948, and 949. The accelerometer is 3 inches south and 3 inches west of grid point # 928.

Global position: (159.4", 204.0", -30.0")

Lower surface Accelerometer "D":

Bounded by points # 723, 724, 744, and 745. The accelerometer is 10 inches north and 3 inches west of grid point # 723.

Global position: (262.4", 81.0.0", -30.0")

EQUIPMENT

ACCELEROMETERS.

All the accelerometers used were PCB ICP Type 353B33 with a nominal sensitivity of 100 mV/g. The upper-surface accelerometers were secured to the deck through a two-stage fastener. First, a 2-inch square, 1/8-inch thick steel plate was glued with epoxy to the deck. The top surfaces of the steel plates were ground to ensure a good surface finish and bonding. An accelerometer mounting stud was then glued to the steel plate with cyanoacrylate ester glue (Super Glue). The accelerometers on the lower deck did not use the steel transition plate. The mounting studs were glued directly to the lower surface of the deck.

EXCITATION AND FORCE GAGE.

The mid-size instrumented modally tuned sledge (PCB Type 086C20) provided the excitation. The hammer was used with the gray tip, this being the third softest of the four available tips. This was the same excitation system that was used during the May, 1999 trial.

ANALYZER.

The analyzer was an Oros, Inc., 16-channel PC-PACK analyzer. The analyzer incorporates ICP signal conditioning and has a very high dynamic range (90 dB per channel). For this project, only the first five channels were used: Channel one for the hammer, and channels two through five for accelerometers "A" through "D".

The manufacturer's calibration specifications for the accelerometers were entered into the analyzer. The hammer sensitivity was entered as 1V/N. This is consistent with the May, 1999 trial. Therefore, the results are correct for relative magnitude (mode shapes), natural frequencies and damping ratios. However, they are not suitable for absolute values of frequency response function.

DATA CAPTURE

Based on the successful experience of the May, 1999 trial, each coordinate was impacted two times, and the frequency response functions were frequency averaged. Care was taken to repeat the data capture for a particular coordinate if there was the slightest doubt as to data quality.

The hammer input range was kept fixed at 1 Volt. The accelerometer input ranges were predominantly 310 mV, with the range being increased to 1 Volt when overloads were detected. Auto rejection of overloaded signals was enabled throughout.

DATA CAPTURE OF UPPER SURFACE – FIRST DATA SET.

The first data capture for the upper surface was from 1110 to 1415 on Monday June 5, 2000. This was a period of 3 hours, 5 minutes, and represented 3.04 measurements per minute. During this period, some time was spent training personnel on different tasks. Thus, data acquisition was slightly slower than the maximum possible with a well-trained crew.

Data were captured in the frequency range 0-1 kHz, with a frequency resolution of 0.625 Hz. This gave a real-time measurement of 1.6 seconds per impact. The settling time was negligible, since the structure had almost completed its ring-down at the end of the 1.6 second measurement time. The time constant for the exponential window for the response channels was set as dictated by the optimization routine included in the "tce" suite of programs, Reference 2, to 0.300 seconds. During the May 1999 trial, a time constant of 0.564 seconds was used. The constant used for this trial represents a slightly sharper window and is probably indicative of a better signal-to-noise ratio for the June, 2000 inspection.

DATA CAPTURE OF BOTH THE UPPER AND LOWER SURFACES – SECOND DATA SET.

The second data capture was for both the upper and lower surfaces. The lower surface data were captured from 0826 to 1119 on Tuesday, June 6. This represents 2.96 measurements per minute. Data capture for the upper surface was repeated in order to obtain a complete set of data for the entire bridge that was captured in a single day. This second measurement of the upper surface was from 1126 to 1400 on Tuesday June 6. This represents 3.72 measurements per minute.

It was raining heavily during the capture of data from the lower surface. However, the rain stopped in time for the upper surface data capture. It did, though, remain cool and windy.

For the Tuesday data, the analyzer settings were exactly the same as for the Monday trial. Data were captured in the frequency range 0-1 kHz, with a frequency resolution of 0.625 Hz and a real-time measurement of 1.6 seconds per impact. The exponential window was again set at 0.300 seconds.

DATA QUALITY

Overall, the data obtained for this trial are of a remarkably high quality. In-field quality monitoring was predominantly with the coherence function. Up to approximately 400 Hz of the coherence function was typically at 98% or better, with virtually no drop-outs. As is expected for impact-generated data, there was a drop-out in excitation power (and hence coherence and data quality). The first drop-out was near 600 Hz. Because of this drop-out, the data above about 500 Hz have a higher signal noise than the data below 500 Hz. However, the high frequency data quality is still acceptable for many applications.

Figure E.5 shows the coherence functions averaged for all measurements and all accelerometers for various sub-sets of the data. For comparison, Fig. E.6 shows the average coherence for the May, 1999 trial. 100% coherence represents "perfect" data. Typically for large-scale vibration testing, anything over about 80% is deemed acceptable. As can be seen from the figure, the overall data quality for this project is very high. The data for the under side surface have a slightly lower quality because of the increased difficulty in hitting upwards from a floating boat. Overall, the June 2000 data are of slightly higher quality than the May 1999 data.

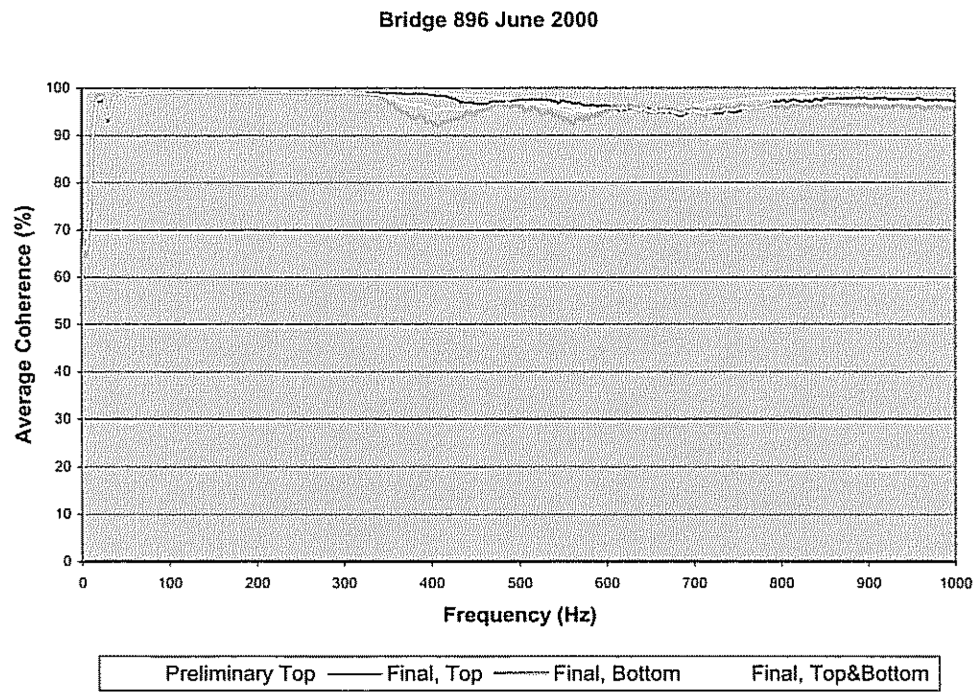


Figure E.5: Averaged Coherence Function for All Accelerometers for June, 2000 testing

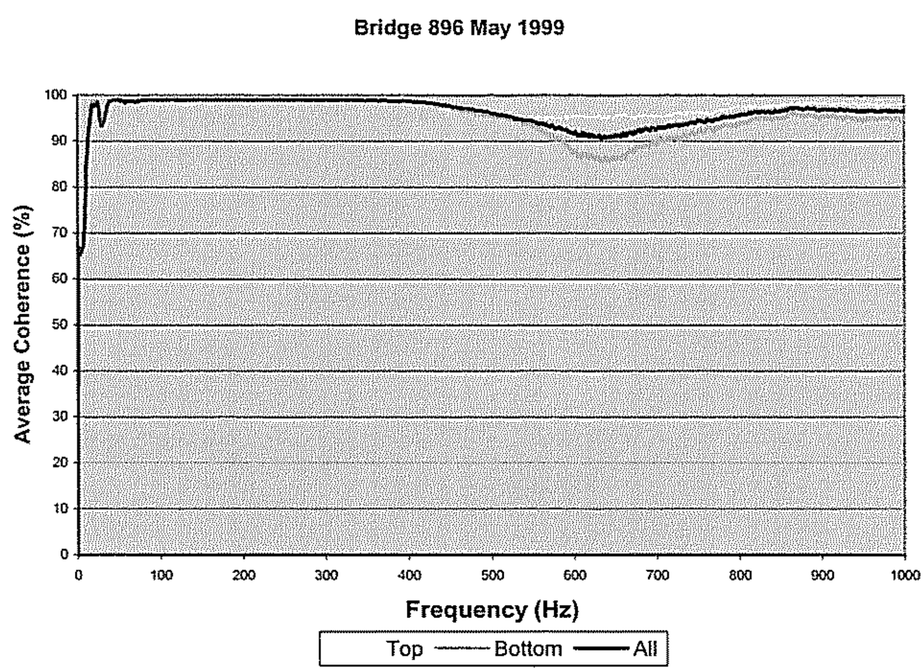


Figure E.6: Averaged Coherence Function for All Accelerometers for May 1999 testing

DATA TRANSLATION

For the May, 1999 trial, data translation from the files captured in Oros AE2 format into a format ready for analysis was conducted using the file translator that is included with the commercial modal software "ME'scope VES" by Vibrant Technology, Inc, Reference 3. This was a very laborious exercise, and required significant manual adjustment of data parameters. The entire procedure took in excess of 30 hours of hands-on time. For this June, 2000 trial, a new "tce" suite of programs, Reference 2, was used to monitor the captured data during the trial, and to translate the data into an analysis-friendly format. The tce suite is tailored for large-scale experimental vibration testing. The entire data translation procedure for the first data set took less than half an hour. Translation of both the first and second data sets took about an hour. This meant the entire provisional data set could be translated on a notepad computer, while traveling home, rather than doing a partial translation of the data in a hotel overnight.

MODAL ANALYSIS

The modal analysis was conducted using the commercial program VES by Vibrant Technology, Inc. Several reference accelerometers were used during the data capture. For this analysis, three different analyses were conducted:

First modal analysis. This analysis was conducted using the data measured from the top surface on the first test day (first data set, Monday June 5, 2000). Accelerometer B was chosen as the single reference accelerometer.

Second modal analysis. This analysis was conducted using the data measured from the entire bridge on the second test day (second data set, Tuesday June 6, 2000). Accelerometer B was chosen as the single reference accelerometer. For this analysis, only the data measured from the top surface was used.

Third modal analysis. This analysis was conducted using the data measured from the entire bridge on the second test day (second data set, Tuesday June 6th, 2000). Accelerometer A was chosen as the single reference accelerometer.

Combined, the analyses determined about 25 modes in the frequency range 18-216 Hz, although not all modes were identified in every analysis. This "missing mode" phenomenon is quite normal for experimental modal work, and is usually attributed to the location of the chosen reference accelerometer.

The natural frequencies and modal viscous damping ratios are presented numerically in Table E.1. The data in this table are shown graphically in Figs E.7 and E.8. The viscous damping ratios have been corrected for the error introduced into the data by the analyzer window constant, Reference 4. The figures and table also include the results from the May, 1999 trial.

Table E.1: Natural Frequencies and Modal Viscous Damping Ratios Both Bridge Vibration Tests

	Natural Frequency (Hz)				Viscous Damping Ratio (%)			
Analysis Number	May 1999	June 2000			May 1999	June 2000		
		Analysis 1	Analysis 2	Analysis 3		Analysis 1	Analysis 2	Analysis 3
1	18.74	18.71	18.84	18.75	4.58	4.38	5.07	4.36
2	24.10	24.84	24.46	24.39	2.58	2.92	2.91	2.86
3	44.67	44.89	44.81	44.83	3.68	3.66	3.98	3.99
4	51.82	53.35	53.05	51.98	1.64	1.74	1.98	1.76
5	76.61	76.49	76.60	76.58	1.33	1.26	1.45	1.46
6		82.39	82.96			1.71	1.73	
7	87.17	87.22	87.07	87.00	1.47	1.54	1.67	1.81
8	90.57	90.26	90.51	90.54	0.83	0.88	0.90	0.88
9	107.51	106.96	107.21	107.20	0.87	0.82	0.92	0.94
10	117.18	116.61	117.09	117.17	1.54	1.59	1.77	1.77
11		120.52	120.68	120.79		1.49	1.63	1.57
12	132.63	131.88	132.27	132.24	0.70	0.74	0.74	0.74
13	136.43	135.95	135.88	136.27	1.33	1.30	1.38	1.49
14	146.83	144.56	145.26	146.19	1.21	0.82	0.87	1.26
15	149.42	148.53	148.51	147.75	1.26	1.61	1.92	1.85
16	155.50	151.52	154.63	155.10	1.24	1.17	1.38	1.29
17	163.34			162.93	1.08			1.45
18	165.87			166.36	0.70			1.30
19	170.54	169.17	170.12	170.19	0.82	1.31	0.89	0.89
20		176.77	177.20	175.45		0.90	0.79	
21	178.36	179.18	179.07	176.94	0.87	0.31	0.90	0.98
22		184.97	184.52	181.05		1.35	0.72	
23		189.01	187.53	188.14		0.75	1.18	0.93
24	207.90	206.63	207.72	206.98	1.11	1.28	1.01	1.10
25	215.97	211.18	212.49	215.44	1.46	1.17	1.21	1.53

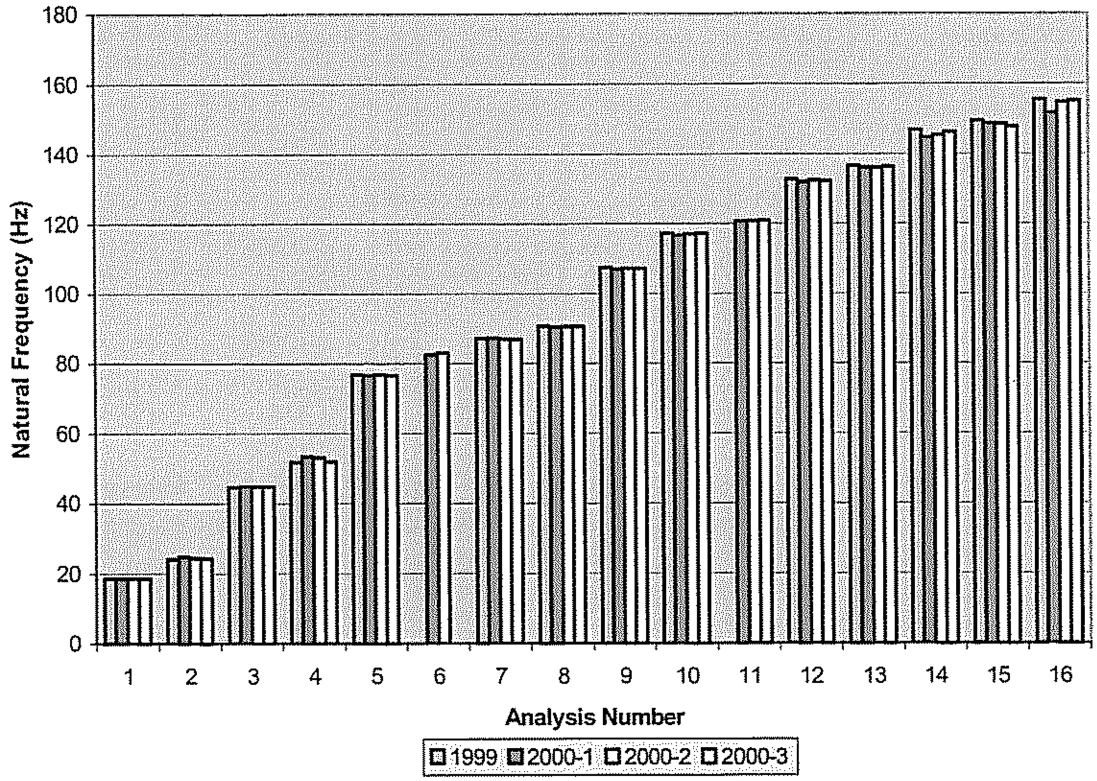


Figure E.7: Natural Frequencies vs. Mode for Both Bridge Vibration Tests

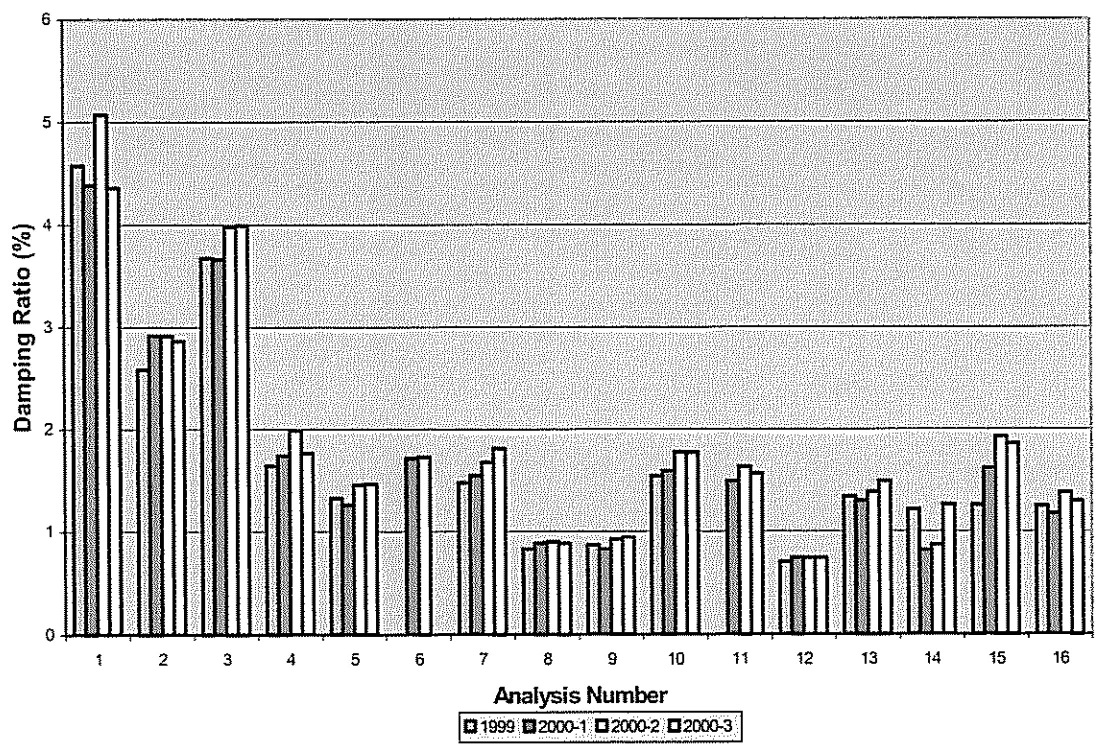


Figure E.8: Modal Viscous Damping Ratio vs. Mode for Both Bridge Vibration Tests

DISCUSSION ON REPEATABILITY

The following observations related to repeatability are made. They are based on a comparison of the analyses done on the data captured on June 5 and June 6, 2000.

a) All mode shapes were smooth and continuous, indicating quality data and analysis.

b) Although not shown in this report, each of the analyses identified the same mode shapes. Despite the dynamic complexity of the bridge at higher frequencies, there were very few "missing modes" from trial to trial.

c) The pooled standard deviation of the natural frequencies is 0.671 Hz. This gives a 95% confidence interval on the natural frequencies of +/- 1.31 Hz.

d) The pooled standard deviation of the viscous damping ratios is 0.1767%. This gives a 95% confidence interval on the damping ratios of +/- 0.345 %.

e) Because of temperature differences, the natural frequencies on Monday were, on average, 0.35 standard deviations lower than those measured on Tuesday. The Monday damping ratios averaged 0.20 standard deviations lower than the Tuesday ratios.

DISCUSSION ON THE EFFECTS OF ONE YEAR

The following observations related to the effects on the modal properties with the passage of one year (383 days) are made. The comparison is between the data measured in May 1999 and the statistics determined from the multiple tests and analyses in June 2000:

With all the comparisons, it must be remembered that the May, 1999 trial was conducted in hot weather (in excess of $80^{O}F$), whereas the June, 2000 trial was in cool weather (mid 60s).

a) All mode shapes measured both years were smooth and continuous, indicating quality data and analysis. The shapes for the June, 2000 trial were, perhaps, slightly smoother than those for the May, 1999 trial.

b) Both years, the west side of the bridge was generally more mobile than the east side.

c) There was some variation in motion between the top and bottom surfaces, especially on the west side. However, this variation was less noticeable in June, 2000.

Figure E.9 through Figure E.12 give representative mode shapes for four of the modes which were also given in the survey of the bridge (1). In each of these figures, the global origin is at the bottom-right of the figure, and north is up-right in the x-direction. The colors in the figures represent amplitude. Red shows the maximum motion; yellow an intermediate motion, down to dark blue, which shows the least motion.

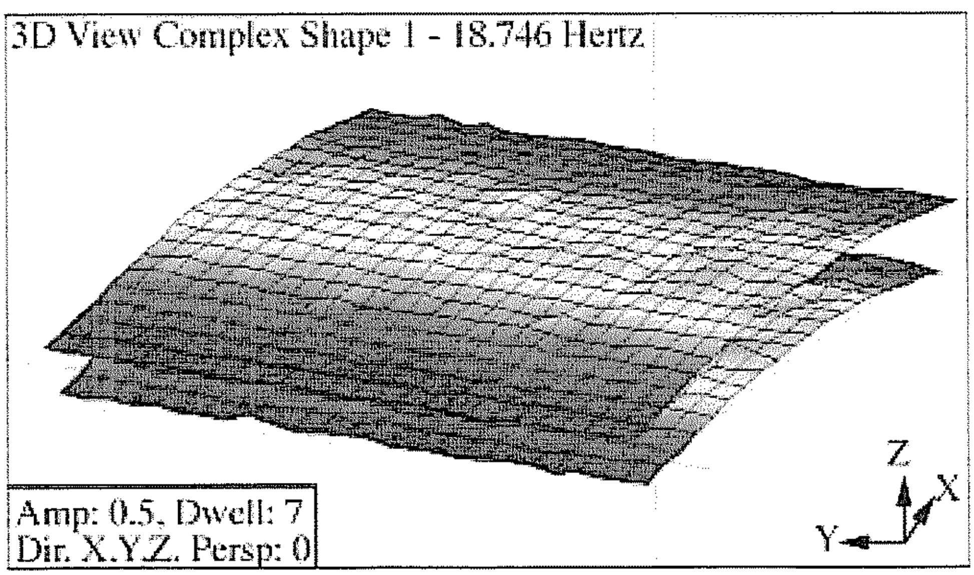


Figure E.9: Mode Shape for Full-scale Bridge Section with 18.746 Hz Natural Frequency

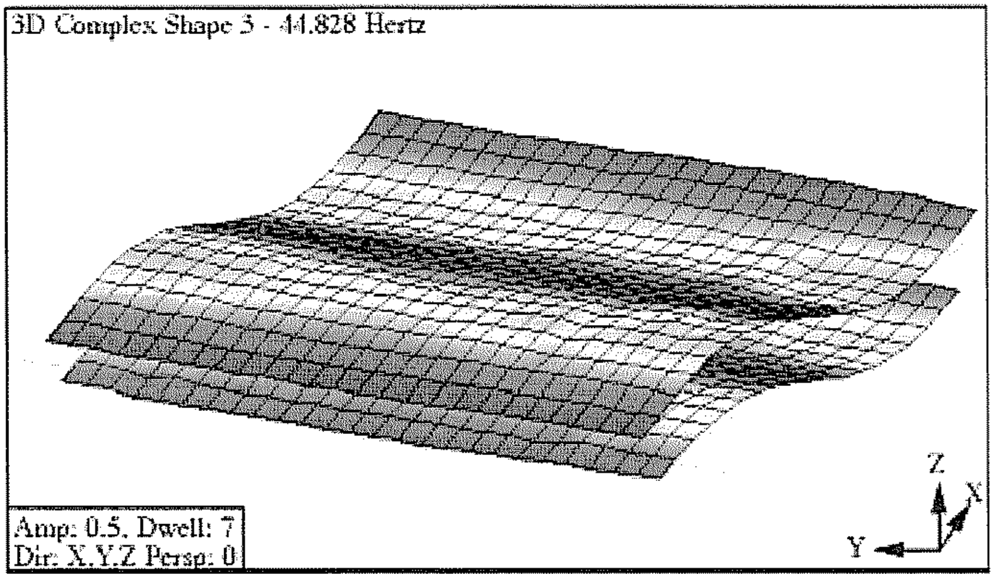


Figure E.10: Mode Shape for Full-scale Bridge Section with 44.828 Hz Natural Frequency

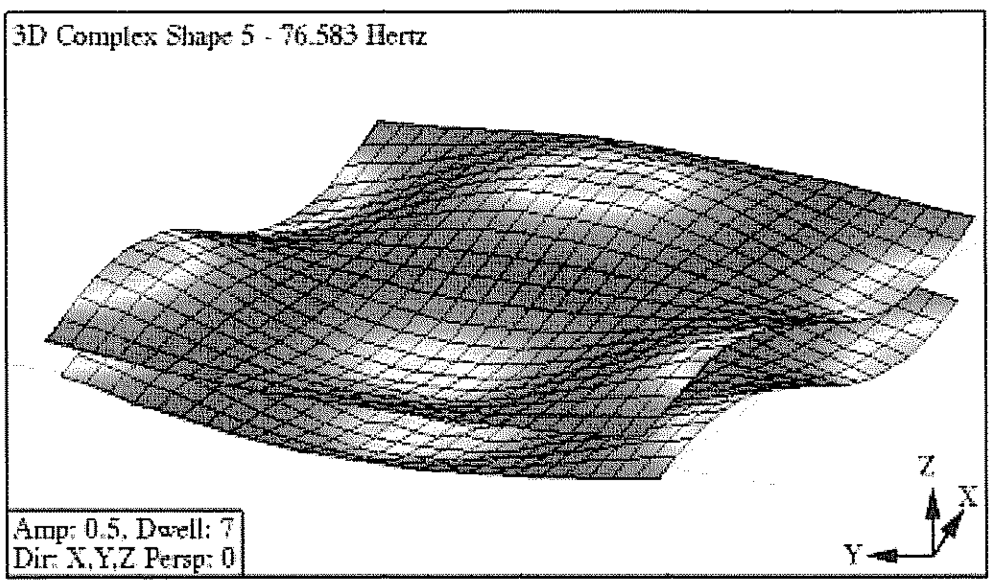


Figure E.11: Mode Shape for Full-scale Bridge Section with 76.583 Hz Natural Frequency

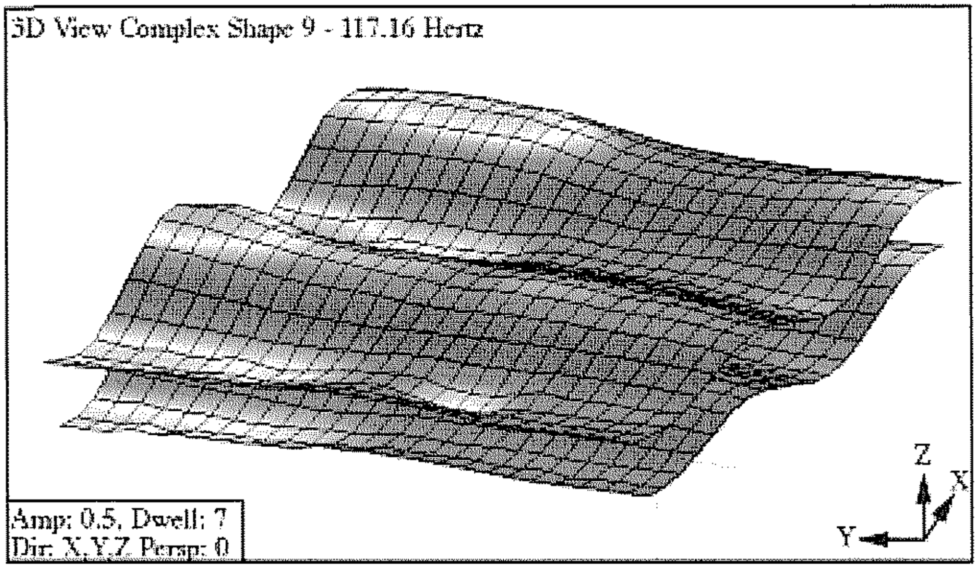


Figure E.12: Mode Shape for Full-scale Bridge Section with 117.16 Hz Natural Frequency

Natural frequencies. Figure E.7 shows a bar chart of all the natural frequencies. Figure E.13 shows the change in natural frequencies. For this figure, the change is determined as the average frequency determined in June, 2000 minus the frequency determined in May, 1999. Thus, a positive value indicates an increasing frequency, and a negative value indicates the natural frequency has decreased. A value of zero represents no change in frequency. This figure includes error bars that show the +/- 95% confidence interval based on the June, 2000 measurements. The conclusion drawn from this figure is that the natural frequencies have not changed with the duration of one year.

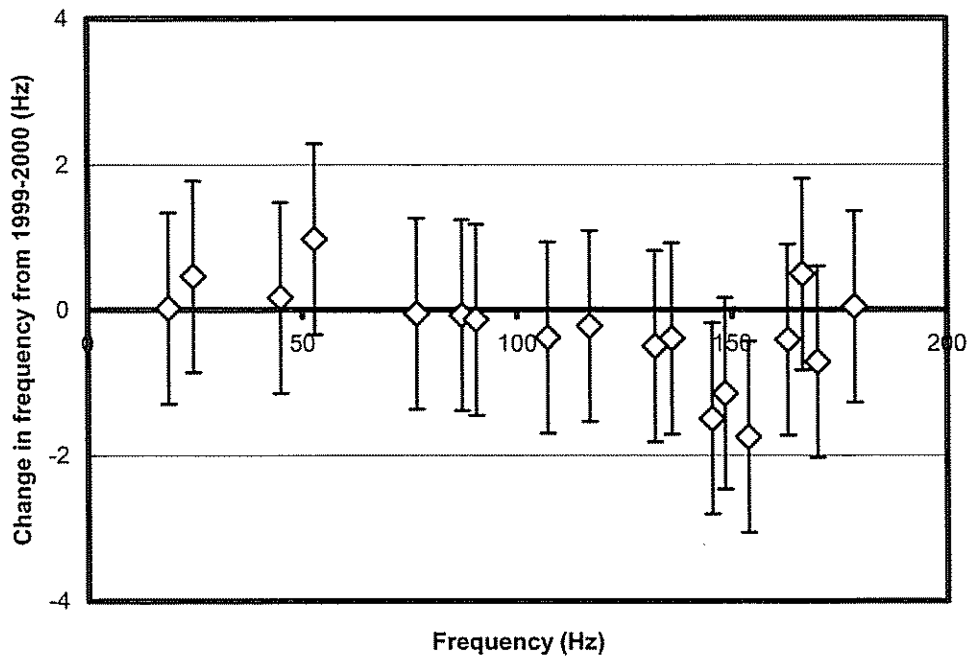


Figure E.13: Change in Natural Frequency for each mode from May 1999 to June 2000

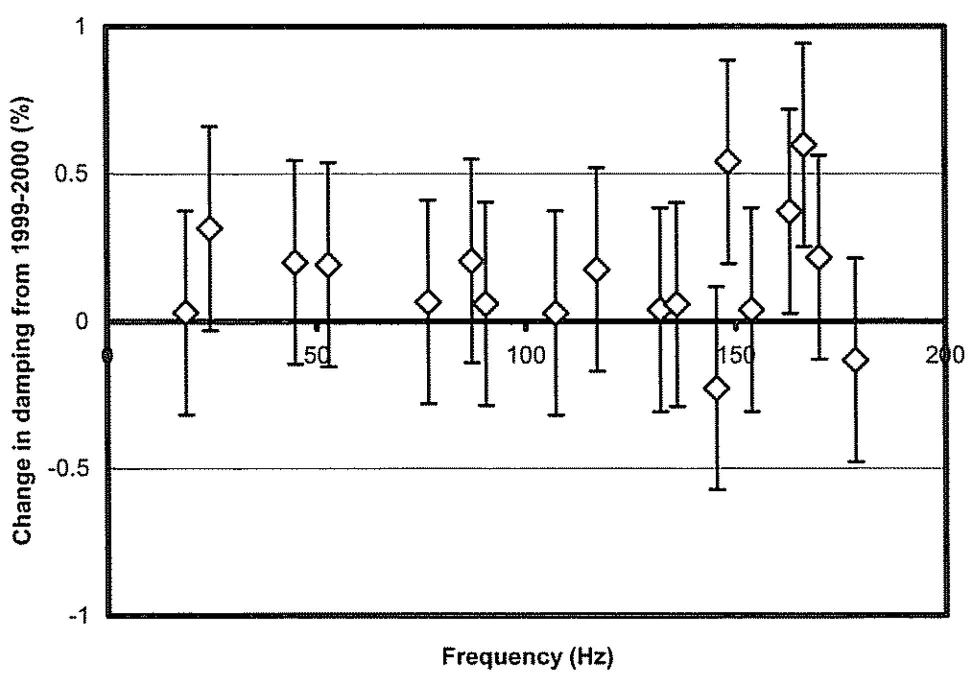


Figure E.14: Change in Viscous Damping Ratio for each mode from May 1999 to June 2000

Viscous damping ratios. Figure E.8 shows a bar chart of all the viscous damping ratios, and Figure 14 shows the change in damping ratios. This figure is determined in the same way that Figure 13 was determined for the natural frequencies. The conclusion drawn from Figure 10 is that, statistically at the 95% confidence level, the damping ratios have not changed with the duration of one year. However, there does appear to be a small bias on the figure, suggesting that overall there might be a small increase in damping, but less than the confidence interval. This increase might be indicative of a build-up of damage (4), or it may be entirely due to measurement and analysis errors.

BROADBAND DAMAGE DETECTION ALGORITHM

The damage detection routine described in this report was not available for the May, 1999 trial. Therefore, this section of the report also presents the 1999 results. This report also presents the comparison between the May, 1999 damage detection results and the June, 2000 results.

To determine the location of manufacturing anomalies in the bridge deck, additional analysis was performed on the vibration data. The technique that was used was a broadband Damage Index Algorithm (DIA), developed by Ratcliffe (5). The method uses frequency-dependent changes in the curvature of the operating deflection shapes as a way of locating damage and other structural irregularities. The method as presented by Ratcliffe (5) operates on beam-like structures (one dimension). In order to apply it to the two-dimensional bridge deck surface, the 1D procedure was applied to each east-west line of coordinates. For each line, the frequency-dependent damage index is plotted as a contour plot. For a spatial position where there is damage or other structural irregularity, the damage index plot will have a feature that is present at all frequencies. Therefore, damage is identified as a feature that is parallel to the frequency axis. An example is shown in Fig. E.15, where damage is showing at about spatial position 16 as the dark region across all frequencies.

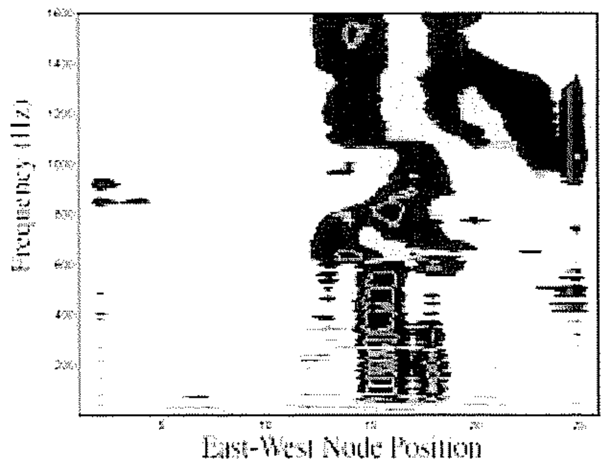


Figure E.15: Results of DIA for one East-West Line of the Test Mesh.

While this method of presenting the results is acceptable for a one-dimensional structure where there is a single accelerometer, the quantity of graphical results mean it is not appropriate for a 2D structure with multiple accelerometers. Instead of looking at one spatial line of coordinates and considering all frequencies, an alternative way of viewing the data is to consider only one frequency at a time, but to look at the entire structure. The Damage Indices for all the east-west lines and for each accelerometer are then combined into an X-Y contour plot. Successive plots can be animated to show how the damage index changes with frequency. For a single-plot summary of the animation, the damage index at each position can be summed across all frequencies. This generates a single contour plot that presents, as a function of position on the structure, the frequency-averaged damage indices.

The results of this summation for the May, 1999 and June 2000, data are shown in Figures 16 and 17. The red areas represent the highest levels of damage index and the blue areas represent the lowest damage indices. For both years, there is a band of higher activity in a roughly north-south line near the center of the bridge. This band is coincident with the center-joining plate, which represents a structural irregularity (increased stiffness). The ability to locate this structural feature is quite significant since the general motion of the bridge deck (mode shapes) did not indicate any anomalous shape feature that would be indicative of this splice plate. Also, it should be noted that the splice plate is covered by a layer of latex-modified concrete.

The natural frequencies and viscous damping ratios indicated that there was probably no significant damage. Similarly, the two Damage Index plots are very similar, suggesting minimal change from 1999 to 2000. As a further refinement, it is interesting to look at the difference in the damage indices from 1999 to 2000. This difference is shown in Figures 18 and 19. Figure E.18 shows a contour plot of the raw difference. While this figure shows the actual difference in the damage indices, the coloration does not easily differentiate between the positive and negative values. While it can be expected that the damage index should increase at areas of increasing damage, it is actually possible for the damage index to decrease over time. This is mainly due to the normalization process that is inherent in the damage detection algorithm. A decreasing index does not mean that the structure is becoming ' undamaged'. Rather, the damage index can reduce in an area that is adjacent to a different area that has increasing damage. Therefore, for informational purposes only, Figure 19 shows the same difference in damage indices, but with negative values artificially set to zero.

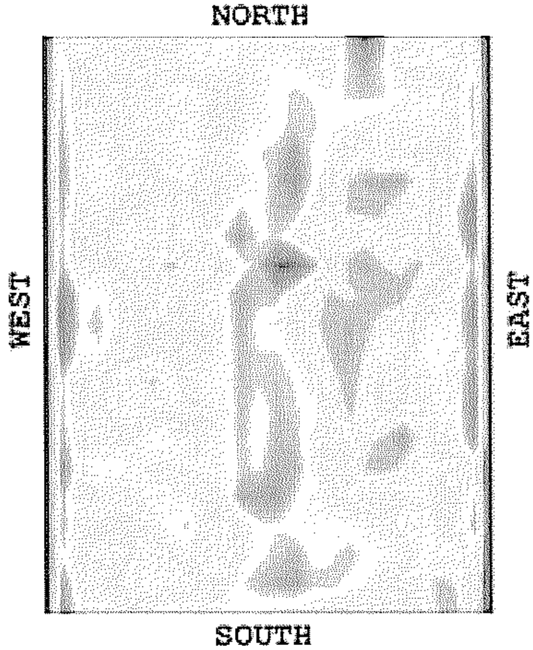


Figure E.16: Damage Index at Each Position Summed Across All Frequencies for May 1999

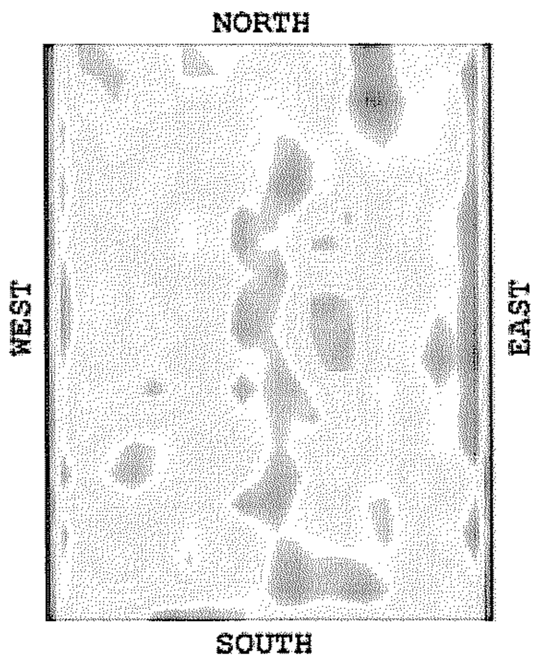


Figure E.17: Damage Index at Each Position Summed Across All Frequencies for June 2000.

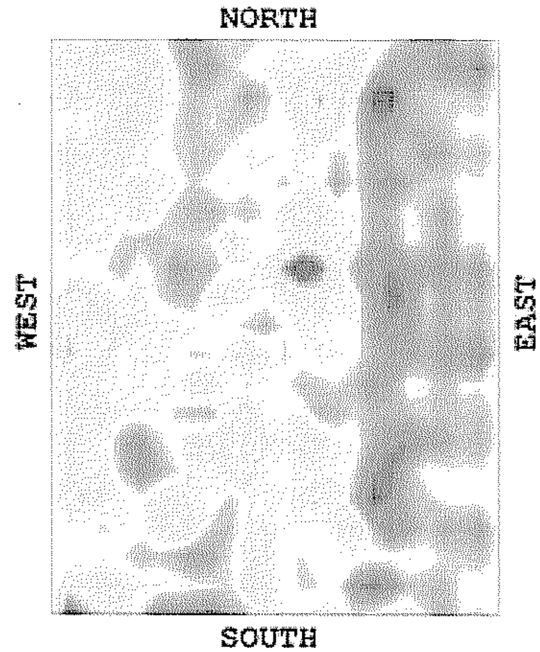


Figure E.18: Difference in Damage Indices from June 2000 to May 1999 Summed Across All Frequencies

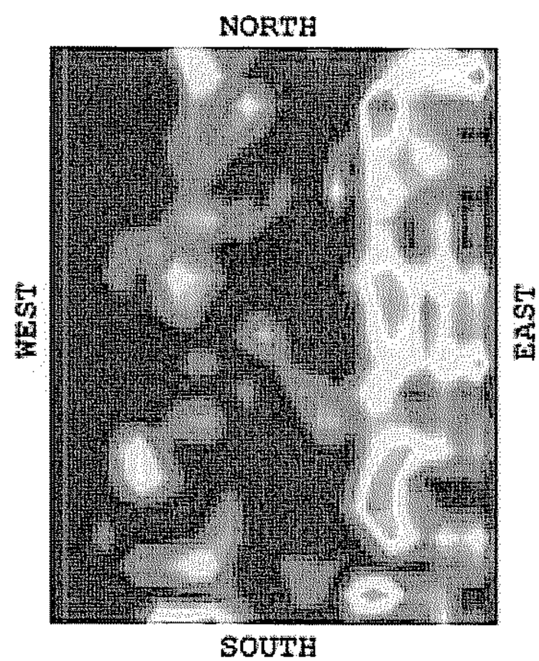


Figure E.19: Difference in Damage Indices from June 2000 to May 1999 Summed Across All Frequencies with Negative Values Artificially Set to Zero

A NOTE ON DATA FILE SIZE

Table E.2 summarizes the approximate disk file sizes for the various data sets. For the trial, there are typically three copies of the data:

AE2 These are the "raw" analyzer data files. They include coordinate number offsets and duplicate copies of data where individual tests were repeated. They are generally not suitable for archiving or regular access for analysis and need "cleaning up" before they can be used.

tce These are "clean" files that contain only the measurement data. They also form the archive of data, and thus are the primary data source for further analysis.

BLK These are specialist data files that hold the frequency response function data in a format compatible with the commercial VES Modal Analysis program. The VES program modifies these files during analysis, and thus they are not suitable for archiving purpose.

Table E.2: Summary of the Approximate Disk File Sizes for the Various Data Sets

File size in MB	Monday June 5th	Tuesday June 6th
AE2 files ("raw" analyzer files)	48.4	87.9
tce files (frf, coherence)	67.8	119.2
BLK files (frf only) (sizes are before/after analysis)	27.1 / 55.4	51.6 / 105.4

CONCLUSIONS

The bridge fundamental resonance was approximately 18.65 Hz in May, 1999. This increased by about 0.1% to 18.76% in June, 2000. Statistically, this change is probably insignificant. From this result it is concluded that the bridge has not experienced a change in its structural performance after one year of service. Since the bridge was designed to have a 75-year service life, these results are encouraging.

The viscous damping ratios showed a statistically insignificant increase. There was, however, a large spread in the results, and the change from 1999 to 2000 was less than the 95%

confidence interval on the data. The increase in damping may be indicative of increased levels of damage, or may be due to measurement and analysis variations. To resolve the issue of damage, a reinspection after another year of service is advisable.

RECOMMENDATIONS

It is recommended that additional annual testing be conducted on the bridge to determine further changes in the overall structural performance. This is the first detailed vibration testing conducted by the authors, and to their knowledge, any other organization in which a statistical assessment could be made regarding changes in frequency and damping occurring over a long period of time. There was statistically no change in the state of the bridge over the year time period.

Further, continued development of the non-destructive damage detection methods, to assess their applicability to locate damage in large-scale structures, is recommended. This has shown promise but also requires validation of the located anomalous structural characteristics by other nondestructive evaluation techniques.

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