The Use of Nondestructive Testing Methods for the Condition Assessment of Concrete Bridge Girders

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ABSTRACT

There are over 594,000 publicly controlled bridges in the United States. Concrete and pre-stressed concrete bridges account for nearly 50% of the bridges in the US inventory. This proportion is increasing each year, as new bridges tend to be constructed of concrete. This trend makes it vital for engineers to be able to accurately assess the condition of concrete for maintenance and repair decisions.

The use of nondestructive testing methods can help reduce the backlog of deficient bridges in two ways. First, these techniques will allow inspectors to get a more accurate view of the condition of a bridge. The second way by which NDT can help is by allowing inspectors to locate damage earlier.

This thesis is an attempt to capture the most current ideas for a very specific application of NDT: determining the condition of reinforced concrete bridges overall and bridge girders, in particular. To this end, attention is given to why NDT is needed and what aspects of concrete condition can be addressed with NDT. Some NDT methodologies that are, or may soon be, promising for concrete applications are discussed. Case studies are presented to demonstrate how NDT can be applied to concrete bridge girders and proposals are made for future areas of study and development.

Thesis Supervisor: Jerome J. Connor
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Acknowledgements & Dedication

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I need to thank my parents, Richard & Dawne Unruh, for their unwavering love and support in all of my endeavors. They have never questioned my path, only helped me to the next milestone.

I want to thank my parents-in-law, Jeff & Mary Smith. They have welcomed me into their family and hearts and provided love and support in all I do. I am fortunate to have two families of which I am proud to be part.

Finally, and most importantly, I dedicate this thesis to my wife, Jenny. Without her love and encouragement, I would not be where I am today. She has given me the strength and courage to achieve goals that I never knew I had. This work is also dedicated to my daughter, Rebecca. She has been an everlasting source of inspiration during my many stressful moments over the past year and a half.
# Table of Contents

ABSTRACT .................................................................................................................. 3
Acknowledgements & Dedication .................................................................................. 5
List of Figures .................................................................................................................. 8
List of Tables .................................................................................................................. 9
1. Introduction ................................................................................................................ 11
   1.1. Scope and Purpose of Thesis .............................................................................. 12
   1.2. State of Bridges in US ....................................................................................... 12
   1.3. How NDT Can Help ......................................................................................... 16
2. Background on NDT .................................................................................................. 19
   2.1. Issues & Barriers to Use of NDT ........................................................................ 19
3. Concerns for Reinforced Concrete Bridge Girders .................................................... 23
   3.1. Rebar Corrosion ............................................................................................... 24
   3.2. Cracking ............................................................................................................ 27
   3.3. Chemical Attack ............................................................................................... 27
   3.4. Honeycombing .................................................................................................. 29
   3.5. Creep, Shrinkage & Volume Changes ............................................................... 29
   3.6. Strength of Concrete ......................................................................................... 31
   3.7. Condition of Post Tensioning Tendons ................................................................ 31
   3.8. Freeze-Thaw Damage ..................................................................................... 31
4. Selected NDT Methods with Applications to Concrete Bridge Girders ....................... 33
   4.1. Pulse Velocity ..................................................................................................... 34
   4.2. Impulse Response .............................................................................................. 35
   4.3. Spectral Analysis of Surface Waves (SASW) ..................................................... 36
   4.4. Modal Testing .................................................................................................... 37
   4.5. Acoustic Emissions Methods ............................................................................ 37
   4.6. Impact Echo ....................................................................................................... 38
   4.7. Fiber Optics ....................................................................................................... 38
   4.8. Galvanostatic Pulse Technique ......................................................................... 39
5. Case Studies ............................................................................................................... 41
   5.1. I-40 Overpass: a Work-in-Progress .................................................................... 41
   5.2. NDT Corporation: Assessment of Concrete Arch Bridges ............................... 49
   5.3. Evaluation of Concrete Quality in a Bridge Deck and Detection of Voids in Tendon Ducts in a Pre-stressed Slab .................................................. 49
6. Future of NDT in Bridge Maintenance ..................................................................... 55
   6.1. Health Monitoring Systems .............................................................................. 55
   6.2. Data Fusion ....................................................................................................... 58
   6.3. Research Needs .................................................................................................. 60
   6.4. Funding Needs .................................................................................................... 62
7. Conclusions ................................................................................................................ 65
   7.1. Benefits of NDT Use with Concrete Bridge Girders ......................................... 65
   7.2. Key Barriers Still to Be Addressed ..................................................................... 65
   7.3. Research Recommendations ............................................................................ 66
References ...................................................................................................................... 68
List of Figures

Figure 1-1: Bridge Proportions by Material Type [Chase & Washer, 1997].............. 11
Figure 1-2: Bridge Age Distribution by Type [Chase & Washer, 1997].................... 14
Figure 1-3: Material Proportions for Bridges, Total and Deficient [Chase & Washer, 1997] .................................................................................................................. 15
Figure 1-4: Deficiency Distribution by Age [Chase & Washer, 1997]..................... 15
Figure 1-5: Location of Structural Deficiencies in Bridges [Chase & Washer, 1997] .................................................................................................................. 16
Figure 2-1: Concrete NDT Usage [Rolander, et al, 2001].................................... 21
Figure 2-2: Changes in NDT Usage over Time [Rolander, et al, 2001]............... 22
Figure 5-1: Sketch of Overpass [adapted from Stubbs, et al, 1999 (2)].................. 41
Figure 5-2: Schematic of Approach Used to Identify Stiffness Properties of Baseline and Existing Structures [Stubbs, et al (2), 1999].......................................... 43
Figure 5-3: Damage Detection Model [Stubbs, et al, 1999 (2)].......................... 43
Figure 5-4: Surface Crack Pattern on the Deck [Stubbs, et al, 1999 (2)]................. 46
Figure 5-5: Comparison of Surface Crack Locations and Damage Localization
   Results from the Field Measurements in December 1997 [Stubbs, et al, 1999 (2)]........ 47
Figure 5-6: Comparison of Surface Crack Locations and Damage Localization
   Results from the Field Measurements in September 1998 [Stubbs, et al, 1999 (2)]........ 47
Figure 5-7: Mobility x Mobility Slope Plot from the Impulse Response Test [Yong Hao, et al, 2003].......................................................... 50
Figure 5-8: Core Sample Showing a Large Void [Yong Hao, et al, 2003]................. 51
Figure 5-9: Impact Echo Test Output for Same Location. Amplitude Spikes Indicate Anomalous Areas [Yong Hao, et al, 2003].................................................. 51
Figure 5-10: Fibroscope View of Void in Tendon Duct [Yong Hao, et al, 2003] ....... 52
Figure 5-11: Impact Echo Output Showing Void in Tendon Duct [Yong Hao, et al, 2003]............................................................................. 53
Figure 6-1: Bridge Management System Flow Chart [Chase & Washer, 1997]...... 56
Figure 6-2: Risk-Informed Expenditure Allocation Flowchart [Ayyub, et al, 2003] 57
Figure 6-3: Factors to Consider in the Development of a Data Fusion System [Gros, 1997] .......................................................................................................... 58
Figure 6-4: Schematic for Development of a NDT Fusion Engine [Gros, 1997]...... 59
Figure 6-5: Design of an Expert System to Assist in NDT Investigations [Gros, 1997]...................................................................................... 60
Figure 6-6: Research Needs for NDT [Rolander, et al, 2001]............................... 61
Figure 6-7: Percentage of Inspections Teams with a PE [Rolander, et al, 2001]....... 63
List of Tables

Table 3-1: Concrete Deterioration Diagnostics [Heckroodt, 2002].............................24
Table 3-2: The Manifestation of Reinforcement Corrosion [Heckroodt, 2002]............25
Table 3-3: Condition Surveys to Evaluate Reinforcement Corrosion [Heckroodt, 2002].......................................................................................................................... 26
Table 5-1: Member Properties [Stubbs, et al, 1999 (2)]...............................................42
Table 5-2: Identified Material Properties of the Baseline Structure (December 1997) [Stubbs, et al, 1999 (2)] ............................................................43
Table 5-3: Predicted Damage Magnitudes (December 1997) [Stubbs, et al, 1999 (2)]...44
Table 5-4: Bending Stiffnesses of the Structure (December 1997) [Stubbs, et al, 1999 (2)] .................................................................................................................... 44
Table 5-5: Identified Material Properties of the Baseline Structure (September 1998) [Stubbs, et al, 1999 (2)] ............................................................45
Table 5-6: Predicted Damage Magnitudes (September 1998) [Stubbs, et al, 1999 (2)]...45
Table 5-7: Bending Stiffnesses of the Structure (September 1998) [Stubbs, et al, 1999 (2)] .................................................................................................................... 46
Table 6-1: Proposed Allocation for Additional Funding [Rolander, et al, 2001] ..........62
Table 6-2: Bridge Inspection Policy & Procedural Change Suggestions [Rolander, et al, 2001] ..................................................................................................................... 64
1. Introduction

There are over 594,000 publicly controlled bridges in the United States. [Better Roads, 2003] Many of these bridges were built in the 1940's and 1950's. As can be seen in Figure 6-1, concrete and prestressed concrete bridges account for nearly 50% of the bridges in the US inventory. This proportion is increasing each year, as new bridges tend to be constructed of concrete. This trend makes it vital for engineers to be able to accurately assess the condition of concrete for maintenance and repair decisions.

![Proportion by Material Type](image)

**Figure 1-1: Bridge Proportions by Material Type [Chase & Washer, 1997]**

Given that most bridges have a design life of 70 years or less, a large portion of our infrastructure will need extensive repair or rehabilitation in the near future. Nondestructive testing (NDT) can play a vital role in these rebuilding efforts by helping to identify the bridges and components that most need attention, thereby enabling maintenance personnel to dispense limited funding in the most efficient manner.
1.1. Scope and Purpose of Thesis

NDT is a dynamic field with frequent developments of new technologies and new uses for existing technologies. As such, any attempt to fully explore the current literature would be outdated almost as soon as it was completed. This thesis is an attempt to capture the most current ideas for a very specific application of NDT: determining the condition of reinforced concrete bridges overall and bridge girders, in particular. To this end, attention is given to why NDT is needed and what aspects of concrete condition can be addressed with NDT. Some NDT methodologies that are, or may soon be, promising for concrete applications are discussed. Case studies are presented to demonstrate how NDT can be applied to concrete bridge girders and proposals are made for future areas of study and development.

1.2. State of Bridges in US

According to the 2001 ASCE “Report Card for America’s Infrastructure”, the US bridge inventory rated a grade of ‘C’, or fair. The reasons for this low rank were a combination of structural deficiencies and functional obsolescence. The Report Card states that, as of 1998, 29% of the bridges in the US were structurally deficient or functionally obsolete. To bring all of these bridges up to date would have cost an estimated $10.6 billion per year for 20 years. [ASCE, 2001] The only positive note provided in the report was that the 29% marked an improvement from 31% in 1996. Popovics noted at this time that simply maintaining the current condition of the bridge inventory and eliminating the backlog would require $80 billion. [Popovics, 2001] In 2003, ASCE released a ‘Progress Report’, with updated figures. In this report, bridges still earned a ‘C’ rating, with a note that “The nation is failing to even maintain the substandard conditions we currently have…” There was incremental improvement (again) in the volume of sub-par bridges, with 27.5% rated as deficient or obsolete. This level would require expenditures of $9.4 billion each year for 20 years. [ASCE, 2003] Better Roads Magazine [Better Roads, 2003] painted a picture that was only slightly better with their inventory of the bridges in the U.S. Overall,
Better Roads found 25.8% of the bridges to be functionally obsolete or structurally deficient. When broken down, this results in 22.3% of all state and federal bridges and 29.1% of local bridges (county, town, city, etc.). Better Roads provided statistics for each state, as well. The states with the worst record were West Virginia (70% of local bridges), Delaware (67% of local bridges), Washington, DC (58% of bridges) and Rhode Island (50% of local, 55% of federal/state). Considering that most of the problems are with locally managed bridges, finding inexpensive and reliable solutions to bridge maintenance are vital. Especially in predominantly rural states, such as West Virginia, the tax base is very low, providing little funding to repair or replace aging bridges. Thus the development of NDT methods to help better target available funds will be beneficial to struggling transportation departments.

Figure 1-2 shows the age distribution for bridges, broken down by material. It is clear from this figure that concrete is gaining steadily on steel as the material of choice for bridges. Steel bridges vastly outnumber concrete now, but relatively few new steel bridges have been constructed since 1975 due to high maintenance costs. The number of new concrete and prestressed concrete bridges has matched or exceeded the number of new steel bridges built since roughly 1965. This trend is expected to continue due to material and maintenance costs.
Steel bridges account for well over half of those rated deficient in the US. Concrete and prestressed concrete account for over 25%, as shown in Figure 1-3. As mentioned previously, however, the number of concrete bridges being built is steadily rising, meaning more bridges will be in need of repair in the future. While steel bridges account for a disproportionate number of deficient bridges, this is primarily due to the age of the structures. Figure 1-4 shows the relationship between structural age and deficiency rating. For bridges built in 1935, or earlier (70+ years old), the proportion of deficient structures rises dramatically. The vast majority of these bridges are steel. Considering an average design life of 75 years and the need for significant overhaul after roughly 40 years, most of the existing concrete bridges either need rehabilitation now or will in the next decade. Not only will the proportion of concrete bridges in need of repair rise in the next 10 to 20 years, but also the overall number of bridges will increase sharply. There have been two major building booms for bridges in the US. The first was immediately after the depression, in the late 1920s and 1930s. The second was during the interstate highway construction in 1950s and 1960s. The majority of deficiencies now are with bridges from the first
boom period. The bridges from the second boom will start to deteriorate in the next few decades.

**Proportion by Material Type**
(excludes culverts and tunnels)

- Other (3493)
- Timber (43446)
- P/S Conc. (34400)
- Concrete (141558)
- Steel (196741)

- All bridges
- Structurally deficient bridges

**Figure 1-3: Material Proportions for Bridges, Total and Deficient [Chase & Washer, 1997]**

**Age Distribution of Structures**

**Figure 1-4: Deficiency Distribution by Age [Chase & Washer, 1997]**
For concrete bridges, the most common reasons for a ‘deficient’ rating are flaws with the substructure followed closely by superstructure (see Figure 1-5, below). These are the most difficult locations to reach for an in-depth inspection. Emerging technologies in NDT and embedded sensing will help inspectors in these areas. Portable sensors and in situ monitoring systems will make it much easier for engineers to assess the condition of key components as well as the overall structure itself.

1.3. How NDT Can Help

The use of nondestructive testing methods can help reduce the backlog of deficient bridges in two ways. First, these techniques will allow inspectors to get a more accurate view of the condition of a bridge. A large number of bridges listed as
deficient are so classified based on a low load rating. The capacity of a bridge is currently determined based on theoretical calculations that may not be accurate. The use of NDT methods such as deflection measurements using laser technology will increase the accuracy of these calculations. [Washer, 1999] NDT also can improve bridge evaluations by reducing the subjectivity of condition assessment. Bridge inspectors tend to base the classification of a bridge on their own experience, leading to high variance in assessments. A bridge may be the worst case one inspector has seen, resulting in a low rating, while another inspector may rate the bridge as ‘fair’. In one such situation, a bridge had received a rating of ‘poor’ by inspectors. Two separate NDT tests were run on the bridge: an impact test and a truck load test. Both tests yielded deflections that were well within AASHTO limits. The bridge was displaying unexpected (and un-designed) composite action between the girders and the deck, giving it a much higher capacity than the inspectors had predicted. [Lennet, et al., 1999]

The second way in which NDT can help is by allowing inspectors to locate damage earlier. Many forms of deterioration, such as reinforcement corrosion, are not visible in their early stages. With corrosion in particular, by the time the problem is visible, extensive structural damage has generally occurred. Technology is available that can detect and evaluate many anomalies that are not visible on the surface of a structure. Ultrasonics, radar and other wave propagation methods all provide inspectors with information regarding the internal flaws in a concrete structure. The reliability of this information is dependent on the nature of the flaw (size, depth and orientation), as well as the testing method itself. Reliability of the various methods will be discussed in Chapters 2 and 4. Other methods can also help will early detection of anomalies. Fiber optics and health monitoring systems with embedded or permanently affixed sensors can also provide information on existing flaws. Locating these problems early will reduce the cost of repairs and increase the reliability and safety of the structure.
2. Background on NDT

In the past, development of NDT methods was driven primarily by military research. From the 1950s through the 1990s, this research was heavily influenced by Cold War concerns, with the benefits being enjoyed across many fields. Since the 1990s, with the end of the Cold War, the research driving NDT development has shifted to private industry, particularly the manufacturing industries. While the automobile and aerospace industries have been using NDT methods for several decades, use of NDT on civil infrastructure projects is a relatively new application. NDT for concrete, in particular, is still a very young field. Significant use of NDT methods for concrete has been occurring only for the last decade or so. The reasons for this lag compared to other industries include both scientific concerns and practical considerations.

2.1. Issues & Barriers to Use of NDT

There are three main barriers to the development of a rational, global NDE method for concrete bridges. [Atkan, et al., 1996] These barriers are:

- Lack of quantitative knowledge of as built state parameters (i.e. initial stresses & strains) and the variation of these parameters over time
- Lack of clear, quantitative definitions for performance parameters
- Lack of a clear and complete understanding of the phenomena which influence the state-of-force in a bridge, which lead to a change of state and/or which lead to a decline in performance

Before NDT methods can be perfected, scientists and engineers must first gain a full understanding of concrete bridge behavior. This includes not only an understanding of the material characteristics of concrete itself, but also an understanding of how stresses are distributed through a bridge structure. This understanding will allow the development of a set of parameters to govern the ideal behavior of a bridge and all of
its components. These parameters can then be used to evaluate the performance of the structure at different stages in its service life.

In addition to the barriers to developing NDT methods, there are obstacles to using the systems in the field. These obstacles can be organized into three areas: accessibility, environment and operator skills. [Prine, 1995]

Accessibility is of particular concern with bridges. Bridge inspections are hampered on several fronts in terms of access. Height, interior access for girders, and the location over water all contribute to accessibility concerns and often result in lowered productivity and reduced reliability. Lifts are often employed to evaluate the superstructure of a bridge. Space limitations in lifts and inside the girders prohibit the use of large equipment. Especially on heavily traveled roadways, lane closures can often be done only during very limited timeframes, forcing inspectors to work at night or around high-speed traffic.

By their nature, bridges are located in harsh environments. They are generally very exposed, leaving inspectors with little protection from high winds and inclement weather. Temperature extremes and precipitation combine with wind to make inspections very challenging and, often, impossible. Vibrations and noise from traffic further inhibit effective inspections. When lane closures are required, nighttime inspections can add the additional hardship of darkness. Rust and dirt often coat connections and members, requiring extensive cleaning and scraping for any technique that requires a direct connection to the structure.

Operator skills are also a limiting factor for field inspections. Especially for firms and agencies with limited budgets, training and certification are difficult to obtain for inspectors. Without training, most inspectors can perform a visual inspection and only very basic NDT techniques, such as sounding. Figure 2-1, on the following page, shows the results of a survey conducted by the Federal Highway Administration on the use of NDT. This figure shows the number of respondents
who indicated they had used each method of NDT. Visual Inspection and mechanical sounding were overwhelmingly the most commonly used NDT methods. Advanced techniques, such as radar, ultrasonics and acoustic emissions saw only very limited use. These methods require more extensive training, which is often unavailable for agencies with tight budgets.

<table>
<thead>
<tr>
<th>Concrete NDE Technique</th>
<th>State DOT</th>
<th>County DOT</th>
<th>Contractors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visual Inspection</td>
<td>38</td>
<td>46</td>
<td>6</td>
</tr>
<tr>
<td>Mechanical Sounding</td>
<td>32</td>
<td>31</td>
<td>4</td>
</tr>
<tr>
<td>Cover Meter</td>
<td>21</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>Rebound Hammer</td>
<td>19</td>
<td>9</td>
<td>2</td>
</tr>
<tr>
<td>Electrical Potential Measurements</td>
<td>11</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>Radar</td>
<td>9</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>Ultrasonics (impact-echo)</td>
<td>8</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>Thermal/Infrared</td>
<td>5</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Acoustic Emission</td>
<td>1</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Vibration Analysis</td>
<td>0</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Radiography</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Ultrasonics (pulse velocity)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 2-1: Concrete NDT Usage [Rolander, et al, 2001]

While Figure 2-1 demonstrates the limited use of NDT, overall usage has been rising in recent years. Figure 2-2 shows the percentages of respondents using NDT in three different surveys. These numbers include all NDT usage, for concrete, steel and timber bridge inspections. Usage increased for all techniques from 1994 to 1998. This trend is evidence that NDT methods are being proven reliable and gaining the trust of the engineering profession. As more research is done to both demonstrate the dependability of NDT and reduce the cost of equipment, usage will continue to rise.
NDE Technique | NDEVC, 1998 | Caltrans, 1994\(^2\) | Rens, et al., 1993\(^4\)
---|---|---|---
Ultrasonic Testing | 81% | 70% | 69%
Liquid Penetrant Testing | 81% | 68% | 25%
Magnetic Particle Testing | 64% | 46% | 40%
Radiographic Testing | 17% | 14% | 12%
Eddy Current Testing | 13% | 3% | 12%

Figure 2-2: Changes in NDT Usage over Time [Rolander, et al., 2001]
3. Concerns for Reinforced Concrete Bridge Girders

Concrete can deteriorate in many ways, due to a multitude of causes. Environment, vehicle collision, poor placement techniques, errors in mix design, chemical reactions, and poor materials can all affect concrete in deleterious ways. Deterioration of concrete is generally due to some combination of causes, further complicating an already difficult inspection process. As shown in Table 3-1, a visual inspection of a structure can give many clues as to the nature of the damage. These clues can help guide the engineer in the selection of appropriate NDT methods to clarify and quantify the extent of the damage.

Some of the indicated deterioration mechanisms are of particular concern with reinforced concrete bridge girders. The girders of a bridge are a vital component of the substructure. Compared to the deck, and even the superstructure, girders are very difficult to access for inspections and repairs. Box girders, in particular, present difficulties due to the problems with inspecting the inside. NDT methods have the potential to make inspections easier and more accurate. With current technology, NDT methods can be used to assess corrosion of rebar, cracking, chemical attacks, voiding and honeycombing, condition of tendon ducts and many other problems.

The following are problems that may arise in any concrete structure, bridge girders included. These issues are selected for discussion for several reasons. The first reason is that these are some of the more common problems that may occur with concrete. The inspection of any structure is likely to produce evidence of several of these issues, at least on a small scale. These problems also either are difficult to detect by visual inspection alone, or the full extent of the problem cannot be determined in this manner. Finally, there are NDT techniques that are available to detect these issues, or have shown the likelihood of being able to do so.
Table 3-1: Concrete Deterioration Diagnostics [Heckroodt, 2002]

<table>
<thead>
<tr>
<th>Visual appearance of deterioration</th>
<th>Type of deterioration and causes</th>
<th>Confirmatory testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large areas of rust stains, cracking along pattern of reinforcement, spalling of cover concrete, delamination of cover concrete</td>
<td>Reinforcement corrosion: exposure to normal climatic conditions, with cyclic wetting and drying</td>
<td>Cover depth of rebar</td>
</tr>
<tr>
<td>Expansive map cracking, restrained cracking following reinforcement, white silica gel at cracks</td>
<td>Alkali-aggregate reaction: concrete made with reactive aggregates</td>
<td>Core analysis for gel and rimming of aggregates</td>
</tr>
<tr>
<td>Deep parallel cracking, pattern reflects reinforcement positions</td>
<td>Drying shrinkage crack: concrete made with reactive aggregates</td>
<td>Petrographic analysis</td>
</tr>
<tr>
<td>Deterioration of surface, salt deposits on surface, cracking caused by internal expansive reactions</td>
<td>Chemical attack: exposure to aggressive waters (e.g. domestic and industrial effluent)</td>
<td>Aggregate testing</td>
</tr>
<tr>
<td>Surface leaching of concrete, exposed aggregate, no salt deposits</td>
<td>Softwater attack: exposure to moving fresh waters (slightly acidic) in conduits</td>
<td>Chemical analysis of water</td>
</tr>
<tr>
<td>Surface discoloration, concrete spalling, buckling, loss of strength, microcracking</td>
<td>Fire damage: exposure to open fires sufficient to cause damage</td>
<td>Core examination for colour variations, steel condition</td>
</tr>
<tr>
<td>Major cracking and localised crushing, excessive deformations and deflections of structural members</td>
<td>Structural damage: structure subjected to overload</td>
<td>Petrographic analysis</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Specialist techniques</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Loading and structural analysis</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Core testing for compressive strength and elastic modulus</td>
</tr>
</tbody>
</table>

3.1. Rebar Corrosion

Corrosion of rebar is the single most common cause of maintenance, rehabilitation or replacement of reinforced concrete structural elements. [Jackson, 1999] [Klinghoffer, et al., 2000] [Heckroodt, 2002] It has been estimated that the cost of repairing damage due to rebar corrosion from de-icing salts alone in the United States is
between 325 and 1000 million Euros ($386 - 1200 million). In the UK, the cost is estimated at 1 billion Euros. Corrosion repairs need to be performed on 10% of the bridges in the UK. [Klinghoffer, et al., 2000] With such high costs to repair corrosion damage, it is vital that engineers have the ability to evaluate the extent of the damage.

Particularly in cold climates or in coastal areas, chloride attack can cause rebar to corrode relatively rapidly. Unlike the bridge deck, however, the damage may not be evident with girders, until the problem has reached the critical stage. Deicing salts wash down from the deck and collect in any cracks or joints in the girders, accelerating the ingress of chlorides. In coastal zones, salt from ocean spray penetrates in the same manner.

Table 3-2: The Manifestation of Reinforcement Corrosion [Heckroodt, 2002]

<table>
<thead>
<tr>
<th>Factor</th>
<th>Influence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geometry of the element</td>
<td>Large-diameter bars at low covers allow easy spalling</td>
</tr>
<tr>
<td>Cover depth</td>
<td>Deep cover may prevent full oxidation of corrosion product</td>
</tr>
<tr>
<td>Moisture condition</td>
<td>Conductive electrolytes encourage well-defined macro-cells</td>
</tr>
<tr>
<td>Age of structure</td>
<td>Rust stains progress to cracking and spalling</td>
</tr>
<tr>
<td>Rebar spacing</td>
<td>Closely spaced bars encourage delamination</td>
</tr>
<tr>
<td>Crack distribution</td>
<td>Cracks may provide low resistance paths to the reinforcement</td>
</tr>
<tr>
<td>Service stresses</td>
<td>Corrosion may be accelerated in highly stressed zones</td>
</tr>
<tr>
<td>Quality of concrete</td>
<td>Severity of damage depends on the concrete quality</td>
</tr>
</tbody>
</table>

Depending on the age and condition of the girders, rebar corrosion can be manifested in several ways, as shown in Table 3-2. Because of the many causes of rebar corrosion and the variety of indicators, it is important that a full survey be completed to ascertain the exact nature, as well as the extent, of the damage present in a
structure. This will allow maintenance personnel to determine the best long-term solution, whether that is repair or replacement. NDT technologies that can assess the extent of rebar corrosion and predict the remaining life of the reinforcement are being developed. Some currently available methods for detecting conditions that can indicate corrosion are listed in Table 3-3. While they all have limitations that must be understood, these methods will prove invaluable to bridge management personnel at all levels. Some of the indicated techniques are described in further detail in Chapter 4.

Table 3-3: Condition Surveys to Evaluate Reinforcement Corrosion [Heckroodt, 2002]

<table>
<thead>
<tr>
<th>Surveys</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visual: use comprehensive checklist</td>
<td>Corrosion during early stages not visible&lt;br&gt;Visual survey first action of detailed investigation</td>
</tr>
<tr>
<td>Delamination: hammer or chain drag</td>
<td>Often underestimate full extent of delamination and internal cracking&lt;br&gt;Not definitive</td>
</tr>
<tr>
<td>Cover survey: use alternating magnetic field to locate position of steel in concrete</td>
<td>Unreliable when:&lt;br&gt;– rebar closely spaced, different types/sizes, at deep cover&lt;br&gt;– site-specific calibrations not done&lt;br&gt;– other magnetic material nearby (windows, bolts, conduits)&lt;br&gt;(Note: austenitic stainless steels are non-magnetic)</td>
</tr>
<tr>
<td>Chloride testing: chemical analysis</td>
<td>Chlorides in aggregates give misleading results&lt;br&gt;Chlorides in cracks or defects difficult to determine accurately&lt;br&gt;Slag, concretes difficult to analyse&lt;br&gt;Large samples required to allow for the presence of aggregates</td>
</tr>
</tbody>
</table>
Carbomnia chemical method (pH indicator)
Slightly underestimates carbonation depth
Difficult to discern colour change caused by pH indicator in dark-coloured concrete
Indicator ineffective at very high pH levels (e.g., after electrochemical re-alkalisation)
Testing must be done only on very freshly exposed concrete surfaces (before atmospheric carbonation occurs)

Rebar potentials: potentiometer (voltmeter) using copper/copper
sulphate reference electrode
Not recommended for carbonation-induced corrosion
Interpretation is a specialist task
Delamination could disrupt potential field, and thus produce false readings
Environmental effects (temperature, humidity) influence potentials
No direct correlation between rebar potential and corrosion rates
Stray currents influence measured potentials

Resistance: Wenner probes and
resistivity meter
Carbonation and wetting fronts affect measurements
Concrete with high contact resistance at surface results in unstable readings
Rebar directly below probe influences readings

Common current linear polarisation resistance (galvanostatic linear
polarisation resistance with guard sensor)
Sophisticated technique, requires considerable expertise to operate
Environmental and material conditions have large influence on measurements and single readings are generally unreliable

3.2. Cracking
There are several reasons why concrete will crack. Causes can include improper pouring or curing methods, water loss during curing, settlement, rebar corrosion, shrinkage and freeze-thaw damage. While some cracking is normal (and impossible to avoid), excessive cracking or the formation of large cracks can impact the strength and durability of the concrete. [NRMCA, 1998] These concerns can be magnified in girders due to the many stresses that are present. The girders in a bridge are subjected to bending forces, shears, axial forces and sometimes torsion. Because of this combination, any weakness in the girders is of utmost concern. NDT can aid with crack detection, especially for internal cracks that are not visible from the exterior of the structure. Monitoring crack formation is important to assessing the overall health of a structure and for determining repair and maintenance needs.

3.3. Chemical Attack
There are three basic mechanisms by which a chemical will affect concrete. The first is a dissolution reaction or “acid attack.” Portland cement is very basic (alkaline),
with a pH between 12 and 12.5. Because of this, it is common for acids suspended in water or air to react with the calcium compounds in the cement paste to form salts and water, following a typical acid-base reaction:

\[
\text{ACID} + \text{BASE} = \text{SALT} + \text{WATER}
\]

If the attacking acids are strong, and highly concentrated, the damage to the concrete can be severe. Under these conditions, the reaction results in material loss, as the decalcification leaves behind a matrix of silica with no binder remaining. [Letourneux, 2001]

The second form of chemical attack is sulphate intrusion. This attack is based on the reaction of the sulphate ion (\(\text{SO}_4^{2-}\)) with components of the cement binder. These reactions form sulphate hydrates such as ettringite and thaumasite. Because these hydrates have larger molecular volumes than the original compounds, there is a net expansion of the cement matrix, causing cracks. Of the two sulphate intrusion mechanisms, ettringite attack is more common. Both modes of attack require the presence of water, with ettringite needing higher temperatures (above 70° F) than thaumasite. Thaumasite attacks almost always involve groundwater with a high sulphate concentration. [Letourneux, 2001] [SCI, 2000]

The final form of chemical attack on concrete is the Alkali-Aggregate Reaction (AAR). Alkali-silica Reaction (ASR) is the most common form of AAR. When the aggregate used in the mix is a silica-based rock (granite, argillite, etc.), alkali pore solution can react with the silica to form a gel that expands with water. If there is enough free water in the matrix, the expansion of the gel will exceed the tensile strength of the concrete, causing cracking. These cracks normally will not be visible until 5 or more years have gone by, but the cracks can expand to more than 1 mm wide over time. [Heckroodt, 2002]
As mentioned in the discussion of rebar corrosion, any situation that can lead to internal deterioration is of particular concern for girders. There are NDT methods that can detect chemical attack on concrete, some of which will be discussed in the next chapter.

3.4. Honeycombing

Honeycombing, and a related problem known as ‘bug holes’, can both be caused by improper vibration or poor formwork. Honeycombing is pockets of coarse aggregate, which can form as the mix settles. This can be a result of poor mix (slump too high, coarse aggregate proportion too high, lack of air entrainment), poor placement technique (lift height too high, delay between lifts too long, more vibration needed), or poor formwork (joints too loose). Honeycombing reduces the strength of the concrete and can also increase porosity and reduce durability. ‘Bug Holes’ are air pockets that form against the formwork. Poor vibration or improper form release agents generally cause these pockets. [Prairie] While bug holes generally do not impact the structural properties of the concrete directly, they can increase the porosity, potentially leading to problems over time. Honeycombing is often impossible to see on the exterior of the member, but there are several NDT methods that can detect problem areas.

3.5. Creep, Shrinkage & Volume Changes

Volume changes in concrete can have many long-term implications, especially with reinforced concrete. During the early curing stage, the concrete is still plastic, so volume changes due to settlement (which can amount to 1% or more) do not cause the formation of large internal stresses. After the concrete cures and becomes more rigid, the reinforcing steel restrains expansion and shrinkage. This restraint induces the formation of tensile stresses within the matrix, which may ultimately lead to cracking. Cracking causes both structural problems and aesthetic concerns. Even during the curing phase, however, the settlement can lead to surface water and water layers under reinforcement if the concrete is not properly finished and vibrated. The
layers of water can result in debonding of rebar and flaking and scaling on the surface. After settlement ends, if the rate of water loss exceeds the rate of bleeding, hydrostatic tension can build up in the pores of the concrete, causing shrinkage. There are three mechanisms that can ultimately lead to plastic shrinkage [Feldman, 1969]:

- The unstable nature of newly formed calcium silicate hydrate results in shrinkage as drying occurs. This phenomenon is not fully understood, but is permanent and irreversible.
- Compressive stresses are set up in the concrete due to menisci forming in the capillaries during drying.
- Energy changes occur at the surface of calcium silicate as the water evaporates.

At least 30% of plastic shrinkage is unrecoverable. In an environment with 50% relative humidity, reaching the equilibrium state for water loss can be accompanied by up to 0.1% volume loss due to shrinkage. As this shrinkage occurs, reinforcement and aggregate resist the volume changes, leading to potentially large internal stresses in the concrete matrix. 30-60% of volume loss due to shrinkage is irreversible. Shrinkage may be increased with the use of certain admixtures, such as water-reducers. If concrete experiences high shrinkage, it is likely also to undergo high creep.

Creep is a long-term effect, which can continue for as long as 30 years. 75% of the creep has been completed after the first year. Creep does not ultimately affect the strength of the concrete. Instead, the creep causes large strains as the concrete adjusts to reduce stresses. This can lead to early achievement of limiting strains, and, subsequently, failure.

NDT can help to detect shrinkage and creep behavior in concrete by identifying indicators of the phenomenon. By using NDT to monitor the stresses, strains and
deformations in a structure, engineers can identify and potentially rectify any structural problems.

3.6. Strength of Concrete
Determining the compressive strength of concrete is important to assessing the condition of a structure. It is also one of the most difficult characteristics to measure. The only reliable method is to take a sample of the concrete (coring) and perform a crush test on it. Even this method is not fully accurate as the properties of concrete can vary dramatically over a short distance. There is currently no reliable NDT method for determining the strength of concrete. [Popovics, 2001] Two methods that have shown promise for estimating the strength are ultrasonic pulse velocity and rebound hammer.

3.7. Condition of Post Tensioning Tendons
There are two main concerns for post-tensioned structures. The tendons must be monitored for corrosion and tension, and the condition of the grout in the tendon ducts must be assessed. These are closely related, as the grout is intended to keep moisture from penetrating the tendons. [Duke, 2002] There are several NDT methods that have been shown to be effective for assessing the condition of both the ducts and the cables. Some of the stress wave methods have been particularly promising.

3.8. Freeze-Thaw Damage
Damage due to freezing is not an issue unless the concrete maintains high free water content after curing. When free water in the pores freezes, it expands by 12%, causing cracking. As the water cycles through freezing and thawing over time, the result is flaking and possibly crumbling of the structure. This problem would be most prevalent in the areas of a bridge structure that hold standing water during the cold months, such as drains (especially clogged), or cracks in the driving surface. Girders may be subjected to this action if there is an accumulation of soil or debris that may
hold moisture on the surface of the concrete. Ducts for post-tensioning cables are also vulnerable, if there is any leakage in the grouting.
4. Selected NDT Methods with Applications to Concrete Bridge Girders

There are essentially two ‘types’ of NDT for concrete. [Malhotra & Carino, 1991] The first category, which focuses mainly on strength properties, would be more accurately labeled as ‘mildly destructive’. These include pullout tests, surface hardness tests and break-off and maturity techniques. The second category measures all the other properties. These methods can measure moisture content, density, thickness and uniformity, among other characteristics. Methods in this category include pulse velocity, stress wave, infrared, and radar methods. This thesis will focus on some of the ‘true’ nondestructive methods from the second category.

The Federal Highway Administration (FHWA) has done extensive research on NDT and the application of these methods to structural health monitoring. [Jackson, 1999] As part of the Strategic Highway Research Program (SHRP), specific methods were identified for use on several issues. These methods were selected based on several criteria:

1. Ability to perform the required function at an acceptable level of performance
2. Simplicity of operation
3. ‘Field ready’ and rugged equipment
4. High degree of reliability

Using these criteria as a guide, a list of techniques was selected for discussion in the following sections. Not all of the methods meet all of these requirements at this time. Those that do not, however, have shown the potential to meet the criteria with further research.
4.1. Pulse Velocity

Much research has been done on the use of the Pulse Velocity (PV) method to evaluate the strength of concrete. Because of the non-homogeneous nature of concrete, there are no physical relationships that directly link pulse velocity and the strength of the material. Research suggests, however, that a link can be found through mathematical modeling methods. Using these concepts, empirical formulas have been found and supported by experimentation. It has been found that PV methods can be a good indicator of early age concrete (few days). [Landis, et al, 1994] This research is still not complete and several factors need further exploration to see the full benefit of this NDT methodology [S. Popovics, 2001]:

- The complexity of the internal structure of concrete
- Factors that affect the strength of concrete may affect pulse velocity differently, especially since the strength of a typical concrete structure is controlled by the strength of the cement paste, whereas the pulse velocity is controlled by the properties of the aggregate
- The insensitivity of the longitudinal pulse velocity to small but important changes in the internal structure of concrete
- The lack of a theoretically justifiable relationship between strength and wave velocity.

The research suggests that these barriers can be overcome by the use of certain simplifying assumptions and a multivariable formulation of the controlling equations. Popovics notes, “...there is no theoretically justifiable relation between strength and pulse velocity even for homogeneous, linearly elastic materials, let alone for concrete. Considerable value can, however, still be derived from formulas for improved nondestructive strength estimation obtained by circumventing the lack of scientific approach and selecting an engineering approach: mathematical modeling.” [Popovics, 2001] Recent advances in technology have also made progress in overcoming the aggregate-defect size issues and attenuation problems. By using multiple transducers, new reconstruction algorithms (SAFT), and
transducer arrays at fixed positions, researchers have had good results in strength evaluation, as well as with void location, crack depth determination and slab thickness measurements. [Kroggel, 2000] An extensive discussion of this research is beyond the scope of this thesis. It is sufficient at this point to state that with further research, it is likely that, using Pulse Velocity techniques, improvement can be made to the current strength estimation methods that yield results with ± 20% accuracy. Further uses of PV may be to qualitatively evaluate the uniformity of concrete and determine the extent and intensity of damage to a structure after an earthquake or other large event. In this way, PV could be used to guide the repair efforts on a structure. [Landis, et al, 1994] [Fáçâouro, 2000]

4.2. Impulse Response

The Impulse Response method is a stress wave test. A hammer with a built-in load cell is used to create a low-strain impact to send stress waves through the member being tested. The response of the structure is measured using a velocity transducer. Fast Fourier Transforms are used to process the time records of both the impact and the response. Plots of dynamic stiffness and mobility and damping are generated from this analysis.

This method has many applications to the evaluation of concrete structures. Conditions that can be detected include [Davis & Petersen, 2003]:

- Voiding caused by slab curling
- Delamination around steel reinforcement
- Low-density concrete (honeycombing) and cracking
- Depth of alkali-silica reaction
- Debonding of asphalt and concrete overlays
- Shear transfer across joints
While there are other methods to detect many of these conditions, IR has the benefits of yielding rapid results through on-site testing, which yields an immediate identification of problem areas. This results in both economic benefits and increased confidence.

4.3. Spectral Analysis of Surface Waves (SASW)

SASW techniques, based on the dispersion variances for different material properties, hold promise in several areas. These methods are particularly useful for characterization of damping and response parameters (elastic profile). During repair operations, the member can be monitored after each step to evaluate the effectiveness of repair efforts. The method is based on a significant velocity change of surface waves (also called Rayleigh waves) in damaged areas of concrete. In tests, there has been excellent consistency between SASW results and visual observations. In addition, SASW revealed more areas that were not evident under visual inspections. Studies have shown potential for SASW to determine the depth of cracking as well.

One advantage to SASW is the need to access only one side of the member, versus the two sides needed for many other wave propagation techniques. If a crack is between the two receivers, a drop in velocity will be seen for all wavelengths, increasing the odds of detection. When a multiple ray wave model is used, the superposition method allows for an exact formulation to be used, with no simplifying assumptions. This results in a more accurate characterization of the system. There are some limitations to the method, however. If the depth of a crack is less than half the depth of the beam from the opposite side, it may not be detected by all of the wavelengths. Also, if the wavelength is more than the depth of the beam, the lamb wave effect may prevent the waves from detecting flaws. There is also a 'threshold' crack area for detection, which is dependent on the wavelength. Cracks smaller than this minimum will not be detected. [Kalinski, 1997] [Landis, et al, 1994] [Popovics & Bystrom, 2000]
4.4. Modal Testing

Like, PV, Modal Testing is a method that shows much promise, but requires further research before it can be fully implemented. Modal testing uses the global response of a structure to an excitation to determine the elastic properties of the structure. When concrete cracks, there is a shift in the structure’s response. The decrease in stiffness due to the crack causes the resonant frequency to decrease, while the modal displacements increase. Tests have been performed using impact hammer and electromagnetic shakers as excitation sources. The shaker has been used with both pseudorandom and swept sine signals. Processing the response using the Frequency Domain Direct Parameter ID (FDPI) technique, the study team was able to determine eigenfrequencies and modal damping ratios for the test structure. [Ndambi, et al, 2000] Because the eigenfrequencies are very sensitive to any change in stiffness, this is a very effective method to determine elastic properties of a structure. The method also has the potential to be an indicator of impending failure, given the sensitivity of the slope of the damping versus vibration amplitude to crack damage. While these tests are currently very time consuming and tedious, recent technological improvements are reducing this issue. However, there are still two significant issues with using Modal Testing. The first is the sensitivity of the technique to excitation and data processing methods. The second is how to perform the method in the field, given the need to excite measurable responses in large structures with portable equipment. Further research is needed to resolve these two issues before Modal Testing can be used on a more widespread basis. [Ndambi, et al., 2000] [Visscher, et al, 2000]

4.5. Acoustic Emissions Methods

Acoustic Emissions methods can be used to detect corrosion of reinforcing steel. As the steel corrodes and expands, cracking occurs in the concrete matrix and in the interface between the steel and concrete. This cracking generates stress waves in the structure. These waves can be detected at the surface by various sensors. The
effectiveness of AE methods are limited, however by the high attenuation of concrete and the low intensity of the generated waves. [Landis, et al, 1994]

A quantitative method for using AE has been developed based on a moment tensor approach. This method can be used to evaluate the size, orientation and mode of microcracks in concrete. The location is determined from the arrival time of the first P-wave. The other values are found by deconvoluting the AE signals to find the moment tensors (dipole forces) and evaluating the eigenvalues. [Ouyang, et al, 1992] This method will be very difficult to apply in field situations due to the high level of sophistication.

4.6. Impact Echo
The impact echo method is based on the use of impact generated stress waves that propagate through the test specimen and reflect from internal flaws. Thee flaws can then be located and identified by evaluating the timing, intensity and orientation of the reflected waves. This method is useful for determining the thickness of a member, and for determining the depth of a void or delamination. The effectiveness is limited to larger flaws, however, due to the low frequencies generated by the impact, which cause smaller flaws to blend in with the aggregate. [Landis, et al, 1994] Field studies with IE have shown the method to be effective for locating and determining the extent of honeycombing or voids in box girder walls, performing quality assurance on repairs and locating cracks and other internal damage in post tensioned segmental bridges. [Olson, 1992]

4.7. Fiber Optics
Fiber optic sensors have seen growing use in structural health monitoring. Embedded fiber optics can be used to sense a wide variety of physical perturbations, making them particularly useful for concrete structures. With the ability to detect acoustic, magnetic, and temperature fluctuations, as well as rotational and extensional strains, these systems are ideal for long term monitoring of structures. Fiber optic sensors
have been used to evaluate air content, detect cracks, monitor deflections and measure strain (polarimetry). They have also been applied for monitoring curing and evaluating earthquake damage. Because fiber optics are not affected by most adverse conditions, the sensors are ideal for use in bridges. [Ansari, 1992] [Huston, et al, 1992] Fiber optics are most useful in new bridges, or those undergoing significant retrofit, due to the need to embed the sensors in the concrete.

One type of fiber optic sensors, called Bragg Grating Sensors, has been used in many field studies. These sensors have been shown effective for a variety of applications, including:

- Measuring strains in rebar under tension
- Measuring strains in rebar under bending of a concrete prism
- Evaluating shrinkage and creep

For all of these applications, the researchers noted one important limitation, which is easily overcome. The sensors are very sensitive to temperature fluctuations. There are established methods for accounting for temperature when using Bragg sensors. [Moerman, et al, 1999]

4.8. **Galvanostatic Pulse Technique**

This technique is used to determine the rate of rebar corrosion, allowing the assessment of the current condition, as well as an estimate of the remaining life of the structure. This method is a transient polarization technique. An anodic current pulse is generated galvanostatically on the surface of the structure. This induces a polarization of the reinforcement in the anodic direction compared to the free corrosion potential. A reference electrode records the resulting change of the electrochemical potential of the reinforcement as a function of polarization time. [Klinghoffer, et al., 2000] Researchers have established a relationship between the measured corrosion rate and the expected time to corrosion damage:
- Less than 6 μm/year – no damage predicted
- Between 6 and 30 μm/year – damage predicted in 10-15 years
- Between 30 and 300 μm/year – damage predicted in 2-10 years
- Greater than 300 μm/year – damage predicted in less than 2 years

These estimates are based on a linear change of corrosion rate with time, as well as a uniform corrosion rate over the surface of the rebar. As neither of these assumptions is valid, there is an inherent error in the method. Promising results have been obtained using a correction factor to account for these non-uniformities. Advantages to the method are the rapid results (corrosion rate measurement can be made in less than 10 seconds) and the easy transfer of data to a PC, as well as the ability to predict a structure’s remaining life. Limitations include the aforementioned issues with non-uniformity, as well as the dependence of measurements on environmental conditions such as temperature and humidity. [Klinghoffer, et al., 2000]
5. Case Studies

The following case studies have been selected to demonstrate the effectiveness for NDT for detecting anomalies in reinforced concrete structures. While the cases do not all deal directly with concrete bridge girders, the methods demonstrated all can be applied readily to girders.

5.1. I-40 Overpass: a Work-in-Progress

The following case study has been adapted from “Verification of a Methodology to Nondestructively Evaluate the Structural Properties of Bridges,” [Stubbs, et al, 1999 (2)] All information in this section has been excerpted from that report.

The overpass, shown in Figure 5-1, below, is a reinforced concrete box girder bridge, originally constructed in 1960. It crosses Interstate I-40, a four lane highway, and is oriented North-South. The bridge consists of two spans. The North span is 118 feet long, and the South is 123 feet. The superstructure consists of a seven-foot deep box girder with a 34-foot wide deck (with overhangs). The girder has eight-inch wide webs, spaced eight feet, nine inches apart. The south abutment is called Abutment #1; the North is Abutment #2. Bent #2 supports the bridge approximately at the mid-span. The bent is a five-foot diameter column on a spread footing, which rests on sand. The abutments are essentially end diaphragms supported by beams resting on strip footings. Member properties for the bridge are given in Table 5-1.

Figure 5-1: Sketch of Overpass [adapted from Stubbs, et al, 1999 (2)]
Table 5-1: Member Properties [Stubbs, et al, 1999 (2)]

<table>
<thead>
<tr>
<th>Concrete Box Girder</th>
<th>Member No.</th>
<th>$I_{xy}$ (ft$^4$)</th>
<th>$I_{xz}$ (ft$^4$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4-23, 50-69</td>
<td>347</td>
<td>2877</td>
</tr>
<tr>
<td></td>
<td>1-3, 24-26, 47-49, 70-72</td>
<td>354</td>
<td>3084</td>
</tr>
<tr>
<td></td>
<td>27-29, 44-46</td>
<td>362</td>
<td>3293</td>
</tr>
<tr>
<td></td>
<td>30-32, 41-43</td>
<td>370</td>
<td>3500</td>
</tr>
<tr>
<td></td>
<td>33-35, 38-40</td>
<td>374</td>
<td>3605</td>
</tr>
<tr>
<td></td>
<td>36, 37</td>
<td>750</td>
<td>10551</td>
</tr>
<tr>
<td>Column</td>
<td>73-82</td>
<td>31</td>
<td>-</td>
</tr>
</tbody>
</table>

The bridge was initially surveyed on December 22, 1997. Figure 5-2 provides a schematic representation of the approach used to evaluate the bridge. The research team used the modal parameters of the existing structure, information gained from the as-built plans and a sensitivity-based systems identification (SID) procedure to determine the modal parameters for an idealized structure. The two sets of modal parameters (ideal and existing) are used with the Damage Index Method to predict the possible locations and severity of damage in the existing structure relative to the idealized model. The Damage Index Method (DIM) provides information on the stiffness properties of a structure using modal strain energy stored in the pre- and post-damaged models. The method can be used even when no baseline modal data are available. Stiffness properties of the existing structure are determined from the information generated by the DIM.

The first modal test, performed on December 22, 1997, provided an initial estimate of material properties to use as a baseline. These properties are listed in Table 5-2. It is beyond the scope of this thesis to detail the entire analysis procedure used to determine these properties. Details of the procedure can be found in [Bolton, et al, 1999] and [Stubbs, et al, 1999 (1)].
Figure 5-2: Schematic of Approach Used to Identify Stiffness Properties of Baseline and Existing Structures [Stubbs, et al (2), 1999]

Table 5-2: Identified Material Properties of the Baseline Structure (December 1997) [Stubbs, et al, 1999 (2)]

<table>
<thead>
<tr>
<th>Group 1</th>
<th>Group 2</th>
<th>Group 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Deck)</td>
<td>(Column)</td>
<td>k&lt;sub&gt;Foot&lt;/sub&gt;</td>
</tr>
<tr>
<td>E (lb/ft&lt;sup&gt;2&lt;/sup&gt;)</td>
<td>457.12 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
<td>361.02 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
</tr>
<tr>
<td>(3174 ksi)</td>
<td>(2507 ksi)</td>
<td>-</td>
</tr>
<tr>
<td>k (lb/ft)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>38.28 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
<td></td>
<td>38.28 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

Figure 5-3: Damage Detection Model [Stubbs, et al, 1999 (2)]
A finite element model of the structure was created (Figure 5-3) and used to predict the location and severity of damage. The model has 84 elements. 1 through 72 model the deck; 73 through 82 are for Bent #2, and 83 and 84 model Abutments #1 and #3, respectively. Member properties are given in Table 5-1 (above), with the calculated material properties given in Table 5-2. Predictions for damage locations and severity are given in Table 5-3. These predictions were made using the five lowest modes of the structure. These predictions were then used to calculate the bending stiffness values for the members, which are summarized in Table 5-4.

Table 5-3: Predicted Damage Magnitudes (December 1997) [Stubbs, et al, 1999 (2)]

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Damage Severity, $a_i$</th>
<th>about Y-axis</th>
<th>about Z-axis</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>-0.34</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>-0.30</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>48</td>
<td>-</td>
<td>-0.35</td>
<td></td>
</tr>
<tr>
<td>49</td>
<td>-</td>
<td>-0.34</td>
<td></td>
</tr>
<tr>
<td>59</td>
<td>-0.21</td>
<td>-0.34</td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>-0.26</td>
<td>-0.40</td>
<td></td>
</tr>
<tr>
<td>61</td>
<td>-0.28</td>
<td>-0.40</td>
<td></td>
</tr>
<tr>
<td>62</td>
<td>-0.22</td>
<td>-0.37</td>
<td></td>
</tr>
</tbody>
</table>

Table 5-4: Bending Stiffnesses of the Structure (December 1997) [Stubbs, et al, 1999 (2)]

<table>
<thead>
<tr>
<th>Member No.</th>
<th>$E_{ly}$ (lb ft$^2$)</th>
<th>$E_{lz}$ (lb ft$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4-11, 14-23, 50-58, 63-69</td>
<td>1.5862 x 10$^{11}$</td>
<td>1.3151 x 10$^{12}$</td>
</tr>
<tr>
<td>1-3, 24-26, 47, 70-72</td>
<td>1.6182 x 10$^{11}$</td>
<td>1.3151 x 10$^{12}$</td>
</tr>
<tr>
<td>27-29, 44-46</td>
<td>1.6548 x 10$^{11}$</td>
<td>1.5053 x 10$^{12}$</td>
</tr>
<tr>
<td>30-32, 41-43</td>
<td>1.6913 x 10$^{11}$</td>
<td>1.5999 x 10$^{12}$</td>
</tr>
<tr>
<td>33-35, 38-40</td>
<td>1.7096 x 10$^{11}$</td>
<td>1.6479 x 10$^{12}$</td>
</tr>
<tr>
<td>36, 37</td>
<td>3.4284 x 10$^{11}$</td>
<td>4.8231 x 10$^{12}$</td>
</tr>
<tr>
<td>Column</td>
<td>1.1192 x 10$^{10}$</td>
<td>-</td>
</tr>
</tbody>
</table>
The testing sequence was run again on September 26, 1998, with the same accelerometer layout. Table 5-5 provides the material properties determined from that test. Table 5-6 shows the predicted damage locations and magnitudes and Table 5-7 gives the bending stiffnesses of the members.

Table 5-5: Identified Material Properties of the Baseline Structure (September 1998) [Stubbs, et al, 1999 (2)]

<table>
<thead>
<tr>
<th>Group 1 (Deck)</th>
<th>Group 2 (Column)</th>
<th>Group 3</th>
<th>k_{Footing}</th>
<th>k_{Abut+Soil}</th>
</tr>
</thead>
<tbody>
<tr>
<td>E (lb/ft²)</td>
<td>591.88 x 10⁶</td>
<td>254.52 x 10⁶</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(4110 ksi)</td>
<td>(1768 ksi)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>k (lb/ft)</td>
<td>-</td>
<td>-</td>
<td>4.941 x 10⁶</td>
<td>27.97 x 10⁶</td>
</tr>
</tbody>
</table>

Table 5-6: Predicted Damage Magnitudes (September 1998) [Stubbs, et al, 1999 (2)]

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Damage Severity, α_j</th>
<th>about Y-axis</th>
<th>about Z-axis</th>
</tr>
</thead>
<tbody>
<tr>
<td>23</td>
<td>-0.31</td>
<td>-0.40</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>-0.37</td>
<td>-0.44</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>-0.37</td>
<td>-0.44</td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>-0.28</td>
<td>-0.40</td>
<td></td>
</tr>
<tr>
<td>29</td>
<td>-0.36</td>
<td>-0.42</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>-0.41</td>
<td>-0.47</td>
<td></td>
</tr>
<tr>
<td>31</td>
<td>-0.39</td>
<td>-0.48</td>
<td></td>
</tr>
<tr>
<td>73</td>
<td>-0.46</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>74</td>
<td>-0.46</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>
On May 22, 1999, the research team mapped the pattern of surface cracks on the deck of the bridge (Figure 5-4). As can be seen, the patterns tend to run transversely, with the exception of the group of cracks over Bent #2. This pattern was then compared with the predicted damage zones from the 1997 and 1998 evaluations.

Table 5-7: Bending Stiffnesses of the Structure (September 1998) [Stubbs, et al, 1999 (2)]

<table>
<thead>
<tr>
<th>Member No.</th>
<th>( EI_{yy} ) (lb-ft²)</th>
<th>( EI_{zz} ) (lb-ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>1.4171 × 10¹¹</td>
<td>1.0217 × 10¹²</td>
</tr>
<tr>
<td>24</td>
<td>1.3200 × 10¹¹</td>
<td>1.0222 × 10¹²</td>
</tr>
<tr>
<td>25</td>
<td>1.3200 × 10¹¹</td>
<td>1.0222 × 10¹²</td>
</tr>
<tr>
<td>26</td>
<td>1.5086 × 10¹¹</td>
<td>1.0952 × 10¹²</td>
</tr>
<tr>
<td>29</td>
<td>1.3713 × 10¹¹</td>
<td>1.1305 × 10¹²</td>
</tr>
<tr>
<td>30</td>
<td>1.2921 × 10¹¹</td>
<td>1.0979 × 10¹²</td>
</tr>
<tr>
<td>31</td>
<td>1.3359 × 10¹¹</td>
<td>1.0772 × 10¹²</td>
</tr>
<tr>
<td>4-22, 50-69</td>
<td>2.0538 × 10¹¹</td>
<td>1.7028 × 10¹²</td>
</tr>
<tr>
<td>1-3, 47-49,70-72</td>
<td>2.0953 × 10¹¹</td>
<td>1.8254 × 10¹²</td>
</tr>
<tr>
<td>27-28,44-46</td>
<td>2.1426 × 10¹¹</td>
<td>1.9491 × 10¹²</td>
</tr>
<tr>
<td>32, 41-43</td>
<td>2.1900 × 10¹¹</td>
<td>2.0716 × 10¹²</td>
</tr>
<tr>
<td>33-35,38-40</td>
<td>2.2136 × 10¹¹</td>
<td>2.1337 × 10¹²</td>
</tr>
<tr>
<td>36, 37</td>
<td>4.4391 × 10¹¹</td>
<td>6.2449 × 10¹²</td>
</tr>
<tr>
<td>Box Girder</td>
<td></td>
<td></td>
</tr>
<tr>
<td>73, 74</td>
<td>4.2607 × 10⁹</td>
<td>-</td>
</tr>
<tr>
<td>75-82</td>
<td>7.8901 × 10⁹</td>
<td>-</td>
</tr>
<tr>
<td>Column</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5-4: Surface Crack Pattern on the Deck [Stubbs, et al, 1999 (2)]
It can be seen in Figures 5-5 and 5-6 that the crack locations correlated closely with the predicted damage zones from both years. This provides strong evidence to validate the testing procedure used by the team. Further work is underway to establish a correlation of the visible damage with results from pulse echo and Schmidt hammer tests on the structure.

Figure 5-5: Comparison of Surface Crack Locations and Damage Localization Results from the Field Measurements in December 1997 [Stubbs, et al, 1999 (2)]

Figure 5-6: Comparison of Surface Crack Locations and Damage Localization Results from the Field Measurements in September 1998 [Stubbs, et al, 1999 (2)]
5.2. **NDT Corporation: Assessment of Concrete Arch Bridges**

The NDT Corporation is based in Worcester, MA and provides NDT services for a wide variety of infrastructure applications. This particular case relates to the use of sonic/ultrasonic and ground penetrating radar methods to determine the location and spacing of rebar as well as assess the strength of the concrete in arch bridges. The bridges are 80 to 100 years old. The procedure followed was as follows:

- GPR data were acquired along both longitudinal and transverse lines to cover the roadway
- GPR data were also acquired on the underside of the arches
- Sonic/Ultrasonic data are acquired along parallel transverse lines on the underside of the arches
- Results from the GPR tests are used to locate the rebar based on the reflection of the radar signals
- Results from the sonic/ultrasonic tests are used to locate delaminations and concrete thickness based on the frequency of resonance
- Sonic/Ultrasonic data is also used to estimate concrete strength based on compressional velocity
- Transmission velocities of the sonic/ultrasonic tests can also be used to estimate Young’s moduli and Poisson’s ratio of the concrete

The results of the testing are then presented along with visual observations on a plan of the bridge. [NDT Corporation, 2004]

5.3. **Evaluation of Concrete Quality in a Bridge Deck and Detection of Voids in Tendon Ducts in a Pre-stressed Slab**

The following cases are excerpted from “Evaluation of Concrete Structures by Advanced Nondestructive Test Methods – Impact Echo Test, Impulse Response Test and Radar Survey.” [Yong Hao, et al, 2003]
Case 1: Evaluation of Concrete Quality in a Bridge Deck

The structure involved in this case is a pre-stressed segmental concrete bridge. A tendon reportedly slipped during the pre-stressing process, causing a crack in the deck surface near the stressing block area. A series of destructive and nondestructive tests were conducted on the affected area (approximately 4 meters by 4 meters). The tests included:

- Impulse response test
- Impact echo test
- Core samples
  - Compressive strength test
  - Chemical composition analysis
  - Petrography examination

These tests were also run on an undamaged section of concrete from the deck. The results were then compared to evaluate the effectiveness of the NDT methods.

The impulse response test was run first to determine the general locations of any anomalies. Each test point was evaluated based on average mobility and mobility slope. Those areas showing higher values for both criteria were selected for further testing. Figure 5-6 shows the plot from one of the test locations. Flaws can be detected easily with the contrasting colors.

![Figure 5-7: Mobility x Mobility Slope Plot from the Impulse Response Test](image)

Figure 5-7: Mobility x Mobility Slope Plot from the Impulse Response Test [Yong Hao, et al, 2003]
Impact echo tests were performed next at the same locations as impulse response. These results were then superimposed and cores were extracted at the locations identified as damaged. These cores confirmed the results of both NDT tests. The core from a location with a large void is shown in Figure 5-7. The impulse response test at this location showed a very high mobility compared to intact locations. The impact echo test, shown in Figure 5-7, also indicated the presence of anomalies.

Figure 5-8: Core Sample Showing a Large Void [Yong Hao, et al, 2003]

Figure 5-9: Impact Echo Test Output for Same Location. Amplitude Spikes Indicate Anomalous Areas [Yong Hao, et al, 2003]
Case 2: Detection of Voids in Post Tension Tendon Ducts of Pre-stressed Slab

This case presents the results of impact echo tests performed on a pre-stressed slab. The structure involved is an industrial building constructed of post-tensioned pre-stressed slabs and beams. Following construction, a report indicated that the grouting of the tendon ducts was not performed according to specifications, so voids were suspected.

A fibrescope was used to conduct a preliminary inspection via grouting outlet hoses. Voids and partially filled grout were detected in some of the ducts. These ducts were selected for further investigation. Impact Echo tests were performed at .5-meter intervals along the faulty ducts. Tests were also performed on intact ducts for quality control. As expected, the tests indicated the voids. The view of the void from the fibrescope is shown in Figure 5-9, while Figure 5-10 shows the output from the impact echo test. The amplitude spike can be seen where the void is located.

Figure 5-10: Fibrescope View of Void in Tendon Duct [Yong Hao, et al, 2003]
Figure 5-11: Impact Echo Output Showing Void in Tendon Duct [Yong Hao, et al, 2003]
6. Future of NDT in Bridge Maintenance

Nondestructive testing methods are being used more and more for the assessment of bridges. Engineers and maintenance personnel have found the technologies to be useful in many applications and situations. With concrete bridges, in particular, NDT can aid inspectors in getting a clear, complete picture of the condition of the structure, both inside and out. Because of the heterogeneous nature of concrete, visual inspection alone cannot provide a full understanding of the characteristics of a member. Without NDT, it is almost impossible to determine the extent of degradation without using coring or other highly destructive evaluation techniques, which can further damage an already weakened structure. The use of NDT methods will reduce, or even remove, the need to further damage the concrete. Work can be targeted to only those areas needing repair, providing further cost savings. This potential makes NDT an integral component of a bridge maintenance plan.

6.1. Health Monitoring Systems

Structural Health Monitoring can be defined as “...the measurement of the operating and loading environment and the critical responses of a structure to track and evaluate the symptoms of operational incidents, anomalies and/or deterioration or damage indicators that may impact operation, serviceability or safety reliability.” [Aktan, et al, 1999] Effective bridge maintenance plans utilize some form of structural health monitoring system. These systems gather data from multiple sensors attached to, or embedded in, various structural elements. This data is then used to assess the condition of the bridge and make maintenance decisions. Health monitoring systems tend to measure the global condition of the structure. More narrowly focused techniques are then required to identify and classify the problem(s) more precisely and determine a course of action. [Chang & Liu, 2003]

Historically, health monitoring has involved heuristic approaches such as visual inspection. Recent studies have shown that this approach is not effective, prompting
further research into appropriate monitoring schemes and methods. This research has focused on adapting technology currently used for transportation monitoring systems. Closed circuit television, cameras, sensors and other hardware and software designed for monitoring traffic patterns in tunnels and intersections hold great promise in monitoring the structures themselves. The goal of such studies is to develop a plan for an integrated asset management approach to health monitoring. [Atkan, et al, 1999]

As is shown in Figure 6-1, data collection is the first step in a maintenance program. NDT methods provide unbiased, quantitative data that is crucial to effective decision-making. In situ monitoring systems will provide maintenance personnel with real-time data, improving preventive maintenance capabilities.

![Bridge Management System Flow Chart](image)

**Figure 6-1: Bridge Management System Flow Chart [Chase & Washer, 1997]**

Collecting reliable, accurate data is only the first step in a health-monitoring plan. The data then need to be processed and analyzed, and used to make decisions regarding the structure. This analysis needs to be able to provide information about the current condition of the bridge, how that condition compares to some reference condition (as-built data), and what that comparison indicates about the future condition of the bridge. Current research is breaking down the barriers to the first
two requirements. The future research needs in these areas have been discussed in prior sections. The last component is the most difficult part to determine. Predicting the future behavior of a structure is an exercise in probabilities. Extensive data needs to be collected about bridge condition and behavior. Trends need to be extracted from this data to provide engineers with likely scenarios based on given observations. This information can then be used as part of a risk-based allocation system. Such a system will allow maintenance personnel to allocate resources based on the risk of failure for each structure. Those bridges or members that show the highest risk of impending failure will be repaired first, with lower-risk repairs being delayed. Figure 6-2 shows a flowchart for a web-based risk-informed allocation system. The system receives inputs of data from NDT inspections (ASM, MC, CE methods), consequences of failures or delayed maintenance, and system parameters. The output is a recommended resource allocation.

Figure 6-2: Risk-Informed Expenditure Allocation Flowchart [Ayyub, et al, 2003]
The data in a bridge management system frequently comes from multiple sensors, all of which may measure different characteristics in different ways. Combining this disparate data in a meaningful way is an ongoing challenge and has prompted extensive research in data fusion models and methodologies.

6.2. Data Fusion

An individual sensor or testing method can provide only limited information about a structure. Impact Echo, for instance, provides information about internal defects, while eddy current tests are more useful for surface anomalies. Combining the data from two or more such tests can give a better understanding of the entire structure. The idea behind data fusion, then, is to combine data from multiple sources into a meaningful and useful framework. According to X. E. Gros, "Data fusion can be defined as the synergistic use of information from multiple sources in order to assist in the overall understanding of a phenomenon." [Gros, 1997] In order to create a data fusion system that will be useable and efficient, many factors must be considered. These factors, as shown in Figure 6-3, include the location of the sensors, the nature of the sensors and the structural element being evaluated, as well as human factors such as inspector qualifications and experience.

![Figure 6-3: Factors to Consider in the Development of a Data Fusion System [Gros, 1997]](image)

Once the parameters for the system have been identified, a data fusion engine can be developed, based on the schematic in Figure 6-4. In order for this system to be useful, large amounts of data must be gathered and information entered about
previous tests. This data will allow the engine to determine probabilities and compare data to better process the new information.

![Figure 6-4: Schematic for Development of a NDT Fusion Engine [Gros, 1997]](image)

Data fusion can be incorporated into an expert system to further automate NDT investigations (Figure 6-5). This system will utilize fuzzy logic type algorithms to evaluate incoming data. The results of this analysis can then be further processed for failure analyses, and estimations of defect size, location and orientation. [Gros, 1997]
By combining human experience and automated analysis, expert systems can help to reduce human error in the decision-making process. The use of data from multiple sensors provides an improved understanding of the structural health of a bridge, allowing for more informed decisions.

6.3. Research Needs

NDT methods for concrete have improved greatly in the last decade. When compared to NDT technology for other industries, such as aerospace, these improvement pale. Concrete NDT is relatively immature when compared to these other applications. There are many reasons for this lag. The most significant reason is a lack of research. The nature of concrete and it behavior are not well understood. Because concrete is a heterogeneous material, it is often difficult to isolate harmful defects from naturally occurring (and generally harmless) inclusions. In contrast to steel and other metals, there are no defined, universal failure criteria for concrete. Full use of any testing method, destructive or not, will require a better understanding of concrete behavior.
Research is also needed for the techniques themselves. While there are many methods that have shown promise in laboratory settings, many of these will need considerable adaptation before they will be useful in the field. Investigations are needed to identify exactly how concrete responds to the testing methods and what information can be determined from that response. Many of the techniques currently require bulky, or very sensitive, equipment that is unsuitable for field use. Rugged, portable versions of these tools need to be developed for use in field conditions. The sensors for health monitoring systems currently have a very limited lifespan, and are very expensive and difficult to replace, especially if they are embedded in the structure. More durable equipment is needed to provide long-term monitoring for structures.

A survey conducted by the FHWA found the greatest needs to be methods for evaluating prestressed concrete superstructures and concrete decks. [Rolander, et al, 2001] The survey was sent to state and county departments of transportation as well as independent contractors. Figure 6-6 shows the areas of research that respondents felt are critical.

![Figure 6-6: Research Needs for NDT](rolander-et-al-2001)

Figure 6-6: Research Needs for NDT [Rolander, et al, 2001]
The results of this survey question are representative of the trend in bridge construction. As more and more bridges are built using concrete, the need for condition assessment tools and techniques will also grow. The development and implementation of these new technologies will require funding at all levels.

6.4. Funding Needs

Funding is an ongoing need for NDT. Universities, state agencies, private companies and federal agencies such as the FHWA and NSF as well as organizations such as ASCE and ACI all fund research laboratories and provide grant monies. Research alone is not enough, however. Funding is also needed for agencies to purchase equipment and train inspection teams in their use.

Table 6-1: Proposed Allocation for Additional Funding [Rolander, et al, 2001]

<table>
<thead>
<tr>
<th></th>
<th>State DOT</th>
<th>County DOT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Increase use of NDE</td>
<td>15</td>
<td>20</td>
</tr>
<tr>
<td>Increase personnel</td>
<td>15</td>
<td>6</td>
</tr>
<tr>
<td>Increase equipment</td>
<td>14</td>
<td>4</td>
</tr>
<tr>
<td>Improvements to Bridge Management System</td>
<td>12</td>
<td>23</td>
</tr>
<tr>
<td>Increase time per inspection</td>
<td>10</td>
<td>17</td>
</tr>
<tr>
<td>Increase training</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>Maintenance improvements</td>
<td>2</td>
<td>—</td>
</tr>
<tr>
<td>Remote bridge monitoring</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Improve QA/QC</td>
<td>2</td>
<td>—</td>
</tr>
<tr>
<td>Perform inspections in-house</td>
<td>2</td>
<td>—</td>
</tr>
<tr>
<td>Inspect “bridges” shorter than 20 ft</td>
<td>—</td>
<td>1</td>
</tr>
<tr>
<td>Increase scope of scour surveys</td>
<td>—</td>
<td>1</td>
</tr>
<tr>
<td>Improve repair recommendations</td>
<td>—</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 6-1 shows the areas where state and county departments of transportation would spend additional funding for NDT. These responses were obtained as part of an FHWA survey on NDT use in 1998. [Rolander, et al, 2001] For state agencies, more personnel and equipment ranked very highly. Improvements to a management system are the highest priority for county agencies. These improvements would likely be in the area of data handling and resource allocation. Increasing the use of
NDT methods and allotting additional time for each inspection were also listed as priorities for both state and county departments. All of these changes will require additional funding.

As bridges, and the methods for assessing them, become more complex, inspection teams will need more training. As part of the FHWA survey, respondents were asked about the training required of team members at their organization. Figure 6-8 shows the frequency that a licensed engineer (PE) was included on the inspection team. Overwhelmingly, for state and county DOTs, there is no PE present for bridge inspections. Independent contractors, who tend to have comparatively larger budgets, have a PE on site for 83% of inspections. Experienced, knowledgeable personnel are critical to determining the condition of bridges, and evaluating the maintenance needs.

![Figure 6-7: Percentage of Inspections Teams with a PE](Rolander, et al, 2001)

In the same survey, respondents were asked to list changes and improvements that should be made to policies and procedures. A key issue for county agencies is federal funding. Other responses indicated a need for more training for team leaders and
other members, allotting additional time for inspections and better documentation and quality control. Table 6-2 highlights some of the key responses to this question.

Table 6-2: Bridge Inspection Policy & Procedural Change Suggestions [Rolander, et al, 2001]

<table>
<thead>
<tr>
<th>Category</th>
<th>State DOT</th>
<th>County DOT</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bridge Management System (BMS) Issues</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Electronic data from inspections w/direct input into BMS</td>
<td>6</td>
<td>—</td>
</tr>
<tr>
<td>Require element-level inspection data</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Post bridge repair list on Internet</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Devote more time to inspection and inventory management</td>
<td>—</td>
<td>2</td>
</tr>
<tr>
<td><strong>Training/Continuing Education Related</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Continuing education requirements for team leaders</td>
<td>2</td>
<td>—</td>
</tr>
<tr>
<td>Monitor and audit content of NHI course</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Require Bridge Insp. Training Course for other team members</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Hold single-day refresher course more frequently</td>
<td>—</td>
<td>1</td>
</tr>
<tr>
<td>Standardize continuing education requirements</td>
<td>—</td>
<td>1</td>
</tr>
<tr>
<td><strong>Inspection Operation/Procedure Improvements</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Better access for inspection in urban areas</td>
<td>2</td>
<td>—</td>
</tr>
<tr>
<td>Additional field time by bridge maintenance engineers</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Improved procedures for inspection of prestressed concrete</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Fully documented procedures in a Bridge Inspection Policy Manual</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Regulations for scour (not guidelines)</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>4- to 5-year cycle for Fracture-Critical Members and Special Inspection of major bridges</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Statewide Quality Control</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Summertime inspections</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Mandatory inspections for timber bridges more than 30 years old</td>
<td>—</td>
<td>1</td>
</tr>
<tr>
<td>Structure Inventory and Appraisal (SI&amp;A) form changes too quickly, keep same form for a minimum of 3 to 4 years</td>
<td>—</td>
<td>1</td>
</tr>
<tr>
<td>More equipment to check scour conditions</td>
<td>—</td>
<td>1</td>
</tr>
<tr>
<td><strong>Miscellaneous</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pay consultants on a unit basis, not hourly basis</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>More Federal money (contract inspections, more personnel, training, and software)</td>
<td>—</td>
<td>5</td>
</tr>
</tbody>
</table>
7. Conclusions

Concrete continues to be the material of choice for bridge construction. Since the 1970s, new concrete bridges have outnumbered new steel bridges by a significant margin. With the increase in concrete bridges comes a growing need for tools and technology to evaluate the structures for maintenance decisions. While steel behavior is well understood and NDT methods for steel are fairly well established, concrete lags in both areas. There are many potential benefits to the use of NDT with concrete bridge girders. These benefits will be realized with further research.

7.1. Benefits of NDT Use with Concrete Bridge Girders

NDT technology can improve the inspection of concrete bridge girders in several ways. The data obtained from NDT methods is unbiased and, with the use of multiple sensors, relatively complete. NDT technologies are available that can provide information about the size, location and orientation of voids, the condition of reinforcing steel and post-tensioning tendons, and the condition of grout in tendon ducts. Some methods can also predict elastic properties of the concrete such as Young’s moduli and Poisson’s ratio. The Pulse Velocity method has shown promise for predicting compressive strength, as well. When evaluated by an experienced, knowledgeable inspector, this data will allow for a more complete understanding of the behavior and condition of the bridge members than is possible with more traditional inspection methods. With the more portable methods, or embedded sensors, NDT can make the inspection process easier, faster and safer. The use of embedded sensors will provide real-time data that can be incorporated into a bridge management system to streamline maintenance and repair decisions. All of these advantages will lead to the additional benefit of reduced expenses.

7.2. Key Barriers Still to Be Addressed

Before the full benefits of NDT can be realized, there are several barriers to be overcome. Engineers need a more complete understanding of the behavior of...
concrete under service loads. The response of concrete to the various NDT methods needs to be better understood, as well, to allow inspectors to properly interpret the readings obtained from the tests. Capabilities and limitations of the techniques must also be established; documenting what can and cannot be learned from their use. Data is needed on the long-term behavior of bridges, particularly relating to the effects of damage on this behavior. This information can then be used to predict remaining bridge life and possible failure modes. Finally, additional funding is needed to properly staff, equip, and train inspection teams. Financial constraints are consistently cited as an important factor in decisions to implement or expand the use of NDT methods in bridge inspections. This funding is especially urgent for the public transportation agencies that oversee the majority of the bridges in the U.S. Private corporations tend to claim broader use of NDT, as well as more stringent training requirements. Additional funding will allow public agencies to adopt guidelines and policies to ensure accurate and reliable inspections.

### 7.3. Research Recommendations

In order for NDT to become a reliable, functional tool for bridge girder assessment, several areas of research need to be addressed. This research will support the needs addressed in section 7.2. Based on these needs, research recommendations can be broken down into four categories:

- Material behavior under service loads & extreme events
- Material response to NDT tests
- Development of field-ready testing equipment
- Data collection of bridge failure modes for risk assessment

The order in which this research is undertaken is controlled in part by prerequisite needs for knowledge. The material behavior and response to NDT must be understood before equipment needs can be assessed and data collection performed.

Predictions can be made based on current knowledge, but continuing research must be focused to address the validity of any assumptions being made. Solving
equipment needs will require cooperation among civil, electrical and computer
engineers, as well as materials scientists and many other contributors. Data collection
must be an ongoing process. As more data is added to knowledge banks, predictions
of failure risks will become more accurate and assumptions will be verified or
corrected.
References


