CONCRETE BRIDGE DECK ASSESSMENT USING THERMOGRAPHY AND RADAR

by

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at the

MASSACHUSETTS INSTITUTE OF TECHNOLOGY

January 1987

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Signature of Author ____________________________

Department of Civil Engineering
January 15, 1987

Certified by ____________________________

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Accepted by ____________________________

Professor Ole S. Madsen
Chairman, Departmental Graduate Committee

ARCHIVE
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Submitted to the Department of Civil Engineering on January 15, 1987 in partial fulfillment of the requirements for the degree of Master of Science.

Abstract

Traditional methods of deck condition assessment have proven to be unable to detect early stages of deterioration. They are insufficiently reliable for systematic deck maintenance, especially in the case of asphalt overlaid decks. Alternative testing methods have been examined for potential application to deck condition assessment. The two most promising technologies are ground penetrating radar and infrared thermography. However, experiments have not yet been done to establish how these techniques can be applied under field conditions to evaluate in-service decks. A general framework is presented for a systematic experimental program to: demonstrate the range of capabilities of radar and thermography, evaluate the influence of extraneous environmental variables, and establish the abilities and limitations of these test methods for determining the state of concrete bridge decks. The experimental variables are analyzed and the required parametric relationships are identified. A rationale for determination of the desired relationships with laboratory and field testing is presented. The criteria for selection of field test sites is specified and used to develop a preliminary test matrix.

Thesis Supervisor: Kenneth R. Maser
Title: Research Associate, Civil Engineering Department
Acknowledgments

I dedicate this thesis to my husband, Lindsey Spratt.

_ du hebst mich liebend uber mich,
  mein guter Geist, mein bess'res Ich
"Widmung" Opus 25 Schumann

I thank Ken Maser who provided direction and encouragement in frequent, large doses. His interest and support were essential contributions. Without his comments and suggestions, this thesis would not have gelled into a coherent whole. His guidance greatly improved the quality of both the final document and the learning experience of writing it.

Professor Jerome Connor, when my original funding fell short, came to my rescue and found an alternate source of financial support that allowed me to continue work on this thesis. I deeply appreciate his efforts on my behalf. No matter how busy his schedule becomes, he always finds time for the many students who seek his aid and advice.

It was the generous support of the Fannie and John Hertz Foundation that provided my funding during the not inconsiderable task of writing up the final document.

I wish to acknowledge the support of the New England Surface Transportation Infrastructure Consortium which provided me with the opportunity to explore the field of concrete bridge deck assessment.
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Chapter 1

Introduction

1.1 Statement of the Problem

Bridge deck deterioration is a major problem for highway agencies and is one of the leading contributors to the number of deficient bridges in the United States. For this reason, significant bridge rehabilitation efforts have focused on correcting deck deterioration. In spite of these efforts, deterioration of bridge decks continues to take place at a faster rate than repairs are made.

A reliable assessment of the condition of a bridge deck must be available if a cost effective and systematic approach is to be taken for deck rehabilitation. Defects and deterioration must be identified in order to establish priorities for rehabilitation and to decide on the appropriate method of repair or replacement.

Accurate determination of deck condition is complicated by the fact that signs of degradation are not usually visible at the surface until the deterioration is far advanced. The primary cause of deterioration is corrosion of the reinforcing steel, a mechanism which has been accelerated and amplified in recent decades by the widespread use of deicing chemicals. Reinforcement corrosion deterioration is a subsurface problem so the progress of the damage is hidden until the deck actually begins to spall. The difficulty of detecting damage before it is far advanced is compounded if the concrete deck is hidden from view by a bituminous overlay.

Traditional methods of deck condition assessment have proven to be unable to detect early stages of deterioration. They are insufficiently reliable for systematic deck maintenance, especially in the case of overlaid decks. Alternative testing
Methods have been examined for potential application to deck condition assessment. The two most promising technologies are ground penetrating radar and infrared thermography. Studies have shown that these techniques can detect parameters of interest for determining deck condition.

However, experiments have not yet been done to establish the reliability and consistency of these techniques as they would be applied routinely under field conditions to evaluate in-service decks. Before the promise of these techniques can be realized, a systematic experimental program is required to: demonstrate the range of capabilities of the test methods; evaluate the influence of extraneous environmental variables on the test results; and establish the abilities and limitations of the techniques for determining the state of various bridge decks under a variety of field conditions.

1.2 New England Surface Transportation Infrastructure Consortium

The New England Surface Transportation Infrastructure Consortium was recently formed as a mechanism to focus the resources of the region on the development of substantially improved methods for dealing with the common problems of rehabilitation, reconstruction, and operation of the highway system in New England [NESTIC 85a]. The five states of Maine, Massachusetts, New Hampshire, Rhode Island, and Vermont are currently participants in the Consortium. These states must deal with a number of basic and difficult transportation and management issues. One area of shared concern is that of concrete bridge deck deterioration. The consensus is that current techniques for assessing the condition of concrete bridge decks need to be expanded and improved. All five states agree that the benefits of more reliable deck condition assessment would be substantial.
For these reasons, concrete bridge deck condition assessment was chosen as one of the first topics for investigation on the research agenda of the Consortium. The initial development of the research plan which is the subject of this thesis was done as a scoping effort for the Consortium [NESTIC 86]. Since the research plan was begun in this context, specific examples, descriptions of current practice, and statistics which are cited usually apply to the participating states in the New England region.

1.3 Status of the nation's bridges

Bridge deterioration is a major problem for highway agencies throughout the United States. Over 30 percent of the nation's bridges on major highways exhibit a deficiency in structure or function. The size of the problem is staggering. In October, 1984 the cost to meet existing bridge needs was estimated to be $53.9 billion [USDOT 85a]. This amount only addresses the backlog of existing bridge deficiencies and does not include the cost of repairing or replacing bridges which will newly be identified as deficient under normal use and aging.

The National Bridge Inventory contains information on essentially all of the nation's almost 600,000 highway bridges. At the beginning of 1985, about 41 percent or 236,000 of the 574,100 inventoried bridges were classified as deficient and eligible for Highway Bridge Replacement and Rehabilitation Program funds [USDOT 85b]. These bridges have an assigned sufficiency rating below the satisfactory threshold of 80. The sufficiency rating takes into account the following bridge characteristics: structural adequacy and safety; serviceability and functional obsolescence; and essentiality for public use. Those bridges which are structurally deficient are clearly of greater concern than those which are functionally obsolete.
The reasons for classifying a bridge as structurally deficient may be classified as pertaining to one or more of the following: deck, superstructure, substructure, culvert of bridge length, overall structural condition, and waterway adequacy. In order to determine the relative importance of the various factors contributing to structurally deficient bridges, representative data from the National Bridge Inventory as of January 1, 1985 are presented in Table 1-I [USDOT 85b]. During 1984, the number of previously deficient bridges which were improved to the point that they were no longer classified as structurally deficient was 10,605. During the same year, 16,400 bridges were newly classified as structurally deficient. Table 1-I provides the breakdown into one of the six deficiency categories, in terms of both number of bridges and percentage of total, for the 7,368 off-system bridges and the 3,037 Federal-aid system bridges improved during 1984, and for the 10,310 off-system bridges and the 6,132 Federal-aid bridges newly classified as structurally deficient during 1984. Since any one bridge could have multiple deficiencies, the categories do not sum to the totals.

The primary functions of the bridge deck are to provide a riding surface and to transmit the wheel loads to the underlying supporting members. The deck is thus an integral link in the load path. Since the deck provides the riding surface, highway users are extremely sensitive to deck condition. The deck supports the live load directly and distributes it to the floor system, which carries load to the main supporting members, which in turn carry load to the bearings on the abutments or piers, which finally transfer load from the bridge span down to the supporting ground. The deterioration of bridge decks is one of the leading contributors to the number of deficient bridge decks in the United States. An estimated $400 million is needed annually for bridge deck restoration work alone [Manning 86].
Table 1-I: Classification of Structurally Deficient Bridges 1984

<table>
<thead>
<tr>
<th>ROAD SYSTEM</th>
<th>OFF-SYSTEM IMPROVED NUMBER OF BRIDGES</th>
<th>IMPROVED PERCENTAGE</th>
<th>FEDERAL-AID SYSTEM IMPROVED NUMBER OF BRIDGES</th>
<th>IMPROVED PERCENTAGE</th>
<th>OFF-SYSTEM NEWLY DEFICIENT NUMBER OF BRIDGES</th>
<th>PERCENTAGE</th>
<th>FEDERAL-AID SYSTEM NEWLY DEFICIENT NUMBER OF BRIDGES</th>
<th>PERCENTAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL</td>
<td>7,568</td>
<td>--</td>
<td>3,037</td>
<td>10,310</td>
<td>--</td>
<td></td>
<td>6,090</td>
<td>--</td>
</tr>
<tr>
<td>DECK</td>
<td>2,336</td>
<td>31</td>
<td>1,610</td>
<td>4,620</td>
<td>45</td>
<td></td>
<td>5,963</td>
<td>58</td>
</tr>
<tr>
<td>SUPER-STRUCTURE</td>
<td>3,284</td>
<td>43</td>
<td>1,373</td>
<td>8,691</td>
<td>53</td>
<td></td>
<td>4,745</td>
<td>78</td>
</tr>
<tr>
<td>SUB-STRUCTURE</td>
<td>2,458</td>
<td>32</td>
<td>1,389</td>
<td>9,456</td>
<td>46</td>
<td>27</td>
<td>2,554</td>
<td>25</td>
</tr>
<tr>
<td>CULVERT</td>
<td>3,656</td>
<td>4</td>
<td>78</td>
<td>1,524</td>
<td>3</td>
<td></td>
<td>508</td>
<td>8</td>
</tr>
<tr>
<td>WATERWAY ADEQUACY</td>
<td>320</td>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
1.4 Deck condition rating

The National Bridge Inventory includes a deck rating for each bridge. The deck is assigned a rating from 9 to 0, with 9 indicating best condition and 0 indicating worst condition, based on the presence and extent of the following observed condition indicators: spalls, delaminations, electrical potential, and chloride content. Table 1-II [USDOT 79] describes the condition associated with each numeric rating, stating what action is expected to be necessary to ensure safe and satisfactory service. Table 1-III [USDOT 79] presents the criteria for assigning a deck condition rating as recommended by the Federal Highway Administration. Those bridges with deck condition ratings of 5 or lower are candidates for major repair or replacement.

Tables 1-IV [NESTIC 86] and 1-V [NESTIC 86] illustrate the distribution of bridge deck condition ratings for the New England region [NBI 86], first by number of bridges, and then by area of deck. Nearly 17 percent (2,599) of the region's bridge decks are in poor to critical condition, and thus currently require major maintenance work. An additional 14 percent (2,256) will need repair or replacement in the near future.

It is not economically feasible to perform major replacement of all bridge decks which have low condition ratings. Bridge owners must make decisions which will best allocate scarce funds to correct and mitigate the problems of deck deterioration.
Table 1-II: National Bridge Inventory Rating Descriptions

<table>
<thead>
<tr>
<th>RATING</th>
<th>RATING CONDITION DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>NOT APPLICABLE</td>
</tr>
<tr>
<td>9</td>
<td>NEW CONDITION</td>
</tr>
<tr>
<td>8</td>
<td>GOOD CONDITION - NO REPAIRS NEEDED</td>
</tr>
<tr>
<td>7</td>
<td>GENERALLY GOOD CONDITION - POTENTIAL EXISTS FOR MINOR MAINTENANCE</td>
</tr>
<tr>
<td>6</td>
<td>FAIR CONDITION - POTENTIAL EXISTS FOR MAJOR MAINTENANCE</td>
</tr>
<tr>
<td>5</td>
<td>GENERALLY FAIR CONDITION - POTENTIAL EXISTS FOR MINOR REHABILITATION</td>
</tr>
<tr>
<td>4</td>
<td>MARGINAL CONDITION - POTENTIAL EXISTS FOR MAJOR REHABILITATION</td>
</tr>
<tr>
<td>3</td>
<td>POOR CONDITION - REPAIR OR REHABILITATION REQUIRED IMMEDIATELY</td>
</tr>
<tr>
<td>2</td>
<td>CRITICAL CONDITION - THE NEED FOR REPAIR OR REHABILITATION IS URGENT. FACILITY SHOULD BE CLOSED UNTIL THE INDICATED REPAIR IS COMPLETE</td>
</tr>
<tr>
<td>1</td>
<td>CRITICAL CONDITION - FACILITY IS CLOSED. STUDY SHOULD DETERMINE THE FEASIBILITY FOR REPAIR</td>
</tr>
<tr>
<td>0</td>
<td>CRITICAL CONDITION - FACILITY IS CLOSED AND IS BEYOND REPAIR</td>
</tr>
</tbody>
</table>
Table 1-III: Concrete Bridge Deck Condition Ratings

<table>
<thead>
<tr>
<th>DECK EVALUATION</th>
<th>Condition Indicators (% deck area)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category Classification</td>
<td>Rating</td>
</tr>
<tr>
<td>Category #3 Light Deterioration</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>7</td>
</tr>
<tr>
<td>Category #2 Moderate Deterioration</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>5</td>
</tr>
<tr>
<td>Category #1 Extensive Deterioration</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>3</td>
</tr>
<tr>
<td>Structurally Inadequate Deck</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>State</td>
<td>Total All Bridges</td>
</tr>
<tr>
<td>------------------</td>
<td>-------------------</td>
</tr>
<tr>
<td>Massachusetts</td>
<td>4563</td>
</tr>
<tr>
<td>Maine</td>
<td>2297</td>
</tr>
<tr>
<td>New Hampshire</td>
<td>2360</td>
</tr>
<tr>
<td>Rhode Island</td>
<td>675</td>
</tr>
<tr>
<td>Vermont</td>
<td>2502</td>
</tr>
<tr>
<td><strong>Totals</strong></td>
<td><strong>12,397</strong></td>
</tr>
</tbody>
</table>

**Rating Legend**

1. New - Deck Condition Codes 8 and 9
2. Good - Deck Condition Codes 6 and 7
3. Fair - Deck Condition Code 5
4. Poor - Deck Condition Codes 3 and 4
5. Critical - Deck Condition Codes 0, 1 and 2
<table>
<thead>
<tr>
<th>State</th>
<th>Total All Bridges</th>
<th>Area 1000 Sq.Ft.</th>
<th>% of Total in state</th>
<th>Area 1000 Sq.Ft.</th>
<th>% of Total in state</th>
<th>Area 1000 Sq.Ft.</th>
<th>% of Total in state</th>
<th>Area 1000 Sq.Ft.</th>
<th>% of Total in state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massachusetts</td>
<td>37,203</td>
<td>7,707</td>
<td>20.7</td>
<td>19,506</td>
<td>52.5</td>
<td>3,842</td>
<td>10.3</td>
<td>6,148</td>
<td>16.5</td>
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**Rating Legend**

1. New - Deck Condition Codes 8 and 9
2. Good - Deck Condition Codes 6 and 7
3. Fair - Deck Condition Code 5
4. Poor - Deck Condition Codes 3 and 4
5. Critical - Deck Condition Codes 0, 1 and 2
1.5 Need for deck condition data

Rational bridge management decisions cannot be made without sufficiently accurate, detailed and reliable information on deck condition. There are many places in the allocation of bridge maintenance resources where information on deck condition can be used to improve the quality of decisions. These areas include: inspection, preventive maintenance, priority of projects, preparation of bid documents, construction quality, performance of repairs, and prediction of future needs.

If a variety of techniques are available for deck assessment and the capabilities and limitations of the various methods differ appropriately, the frequency and depth of deck inspections can be tailored to suit bridge management needs. As an example, one set of techniques which is rapid and inexpensive would be appropriate for the biennial National Bridge Inventory inspections mandated for all bridges. This standard condition rating inspection lacks the detail necessary from a preconstruction survey. A more in-depth survey would be undertaken in this case, using more refined techniques whose additional expense would be justified for the different use. The data from the biennial inspection would be available to plan and specify the more in-depth survey.

Condition assessment techniques which are capable of detecting the early stages of deck deterioration are of obvious use for scheduling preventive maintenance. If damage were detected early, installation of a new membrane and overlay or other protective measures could be done to halt the problem before the deck condition became seriously degraded.

The availability of accurate deck condition data influences various stages of major deck rehabilitation projects beginning with the selection of candidate decks.
for repair or replacement. The priorities of various projects are set more effectively. It is possible to make a well-founded decision on whether a replacement or repair is better and on which type of repair would be appropriate. The completeness of the construction documents and the accuracy of the bid estimates are increased. The condition data can be used for contract supervision and construction quality control.

Condition data can also be used to get feedback on how well different deck designs and repairs perform in service. Evaluation of the performance of deck components, protective systems, and repair approaches over time under actual field exposures provides valuable information that can improve the effectiveness of future decisions.

Deck condition information can also be used to predict future needs. In the near term, expected work can be budgeted based on the known current condition of the deck inventory. In the longer term, deterioration models can be used to extrapolate from the current inventory. Accurate assessments are necessary to develop, calibrate, and verify these predictive models.

1.6 Difficulties of deck evaluation

Although the benefits of accurate deck condition data are many, it is difficult to evaluate the actual condition of an in-service concrete bridge deck. The main cause of this difficulty is that the deterioration of the deck is usually hidden until the damage to the deck is far advanced. The primary mechanism of deck deterioration, reinforcement corrosion, occurs below the surface. Signs of corrosion are frequently not visible at the surface until late in the process when pieces of the deck spall free. The concrete deck is frequently covered with a wearing surface overlay. In this case, the signs of progressive deterioration are further obscured.
Evaluation is not only complicated by the surface invisibility much damage. The size of the assessment task is formidable. A majority of the nation's roughly 600,000 bridges have concrete decks. Techniques for deck assessment must be relatively rapid and inexpensive to allow reasonably frequent inspection of this large deck inventory.

The heavy use of bridges in urban areas further complicates the inspection issue. Many bridges and elevated roadways in or near cities cannot be closed to facilitate deck inspections without causing major disruptions in traffic flow.

A final factor that makes the assessment of decks challenging is the penalty associated with an unconservative error. The most difficult bridge management task is accurately estimating the amount of concrete that must be removed for a deck rehabilitation [NCHRPS57 79]. Especially for overlaid decks, the estimated extent of deck damage may vary widely from the actual conditions found during construction. Such an occurrence necessitates a request for additional funds substantially beyond those allocated based on the bid estimates. Such an overrun is usually highly visible politically and extremely undesirable for the highway agency involved.

Experience has shown that current practice for bridge deck condition assessment does not provide the data needed for systematic and well-founded bridge management decisions.

1.7 Objectives of this thesis

Since existing practice is unable to cope satisfactorily with deck assessment, alternate approaches need to be explored. The objectives of this thesis are:
-24-

- to identify new methods capable of accurate, economical, and reliable deck assessment
- to outline the work required before these methods can be practically applied
- to layout a plan for performing this work which is prerequisite to the incorporation of these methods into standard use.

1.8 Content of this thesis

This first chapter has presented the motivation for exploring alternate methods for performing concrete bridge deck assessment. There is a wide spectrum of existing testing techniques that could be brought to bear on this problem. Before testing methods can be meaningfully discussed, the subject of the tests must be adequately defined. The mechanisms of concrete deterioration are described in Chapter 2 so that the parameters affecting the degradation process are understood before the various testing methods are discussed.

A survey of current practice establishes a baseline from which the exploration of alternate methods can begin. Current practice for testing and evaluating bridge decks is reviewed in Chapter 3. Traditional inspection techniques are described and the capabilities and limitations of each method are presented.

Non-traditional technologies for deck assessment are critiqued in Chapter 4. Various methods are compared to desired performance criteria to identify promising techniques. Ground penetrating radar and infrared thermography are identified as the testing methods with the best potential for this application. However, experiments have not yet been done to establish how these techniques can be applied under field conditions to evaluate in-service decks.

With the successful identification of promising techniques complete, the
central task of this thesis becomes the design of a research program to evaluate the performance of ground penetrating radar and infrared thermography in determining concrete bridge deck condition when applied to different deck configurations under various field environments. In order to design a test program that will reach this goal, Chapter 5 discusses the physical bases underlying the two methods. Simple physical models of the deck are proposed and used to predict thermal and radar signal response.

With this background in place, Chapter 6 presents the experimental design for the research program. The experimental variables are identified. The desired relationships between variables are established and test methodology is developed. The characteristics of the existing bridge deck population are surveyed and a field site selection matrix is proposed.

A research program to evaluate the performance of radar and thermography must be placed in a broader context if the results are to eventually lead to practically implementable improvements in the task of concrete bridge deck assessment. To provide this broader context, Chapter 7 presents the objectives and approach for an integrated research program for deck condition assessment currently being executed by the New England Surface Transportation Infrastructure Consortium. Chapter 7 concludes with a brief discussion of how the results of the research program could be used for bridge deck management. The steps necessary to include radar and thermography in routine deck assessment surveys are outlined.
Chapter 2

Deterioration mechanisms

2.1 Types of concrete deck deterioration

Deterioration of bridge decks is not new. However, both the extent of the problem and the amount of funds needed to maintain decks have increased dramatically in recent years. Beginning in the late 1950's, bridge deck distress became widely recognized as a severe and growing problem. By 1967 bridge engineers generally identified concrete bridge decks as the structural item requiring the greatest maintenance effort [NCHRPS4 70]. The primary reason for the greatly increased magnitude of the problem is the extensive use of deicing salts to keep bridges free of ice and snow. The chlorides from deicing chemicals significantly accelerate the process of deck deterioration.

The concrete in a bridge deck is exposed to an extremely harsh environment. Weather conditions subject the deck to alternate wetting and drying, frequent freeze-thaw cycling, and severe temperature gradients. Highway policies which strive for bare pavements in winter result in heavy application of deicing salts. The horizontal orientation of the deck accentuates the severity of weather and chloride exposures. Precipitation lying on the flat surface leads to high saturation which greatly amplifies frost damage and also allows chlorides to infiltrate the concrete. Traffic induces high live-load stresses including fatigue and impact. The congestion of reinforcing steel requires highly workable concrete, and the resulting increase in water cement ratio adversely affects durability. Finishing operations and bleeding mean that the poorest quality concrete is in the most critical position, at the slab
surface. The cover over reinforcing bars is usually only 1-1/2 inches. Given all of these adverse factors, it is not surprising that decks frequently deteriorate.

The common types of deck distress are cracking, spalling, and scaling. Cracking in and of itself does not degrade deck performance, but can facilitate more serious forms of distress, most notably spalling. A spall is formed when a section of the top concrete layer breaks free, leaving a depression in the deck. Spalling is usually preceded by the formation of a planar horizontal crack, referred to as a delamination, at the plane of the top rebar mat. Spalling is the most serious defect and is the most difficult to prevent. Scaling is the flaking away of the concrete surface due to the breakdown of the hardened cement paste matrix. It is a less serious form of deterioration since it can be largely prevented by proper air entrainment during construction.

A bonding failure can occur on asphalt-covered decks. This debonding between the overlay and the concrete is not a significant defect in the deck condition. However, the presence of debonding makes it more difficult to identify delamination, a very important defect in the spalling process.

2.2 Cracking

Cracking occurs in concrete because of the inherent material characteristics of low tensile strength coupled with high volumetric changes due to variations in temperature and humidity. Since the tensile strength of concrete is neglected in design, cracking does not lower the computed deck strength. Cracks also are not usually wide enough to degrade riding quality. Therefore, cracking is not a performance defect in and of itself, but may contribute to other forms of deterioration, most notably spalling.
There are various types of cracks, caused by different factors. Cracks may be categorized as transverse, longitudinal, diagonal, pattern, or random [AASHTO 76a].

Transverse cracks, running perpendicular to the roadway, tend to be the dominant form on bridge decks. These cracks usually occur over the transverse, or primary, reinforcement. During placement, differential consolidation results in more settlement between bars than over them. As the deck undergoes plastic, drying, and thermal shrinkage, cracks form over the bars [NCHRPS4 70]. Parameters which accentuate transverse cracking are: low cover; high water/cement ratio; adverse placement conditions; continuous spans; steel superstructure; negative moment areas of concrete bridges; increased span length; and absence of stay-in-place forms [Balduman 83].

Longitudinal cracks run parallel to the roadway. They are caused in much the same way as transverse cracks, but are less severe since they are associated with the secondary reinforcement direction. They occur most frequently in simple spans. They are common between prestressed concrete box beams [AASHTO 76a].

Diagonal cracks tend to be shallow and lie at a non-right angle to the roadway. Their primary cause is drying shrinkage. They are accentuated by superstructure skew.

Pattern or map cracking forms an interconnected network of cracks. Pattern cracking is most commonly due to: bleed channels formed by excessive finishing; drying shrinkage caused by improper curing; or the presence of reactive aggregates.

Random cracking is irregular and does not form any particular pattern on the deck surface. Random cracking forms on younger decks due to drying shrinkage and on older decks as part of the spalling process [Balduman 83].
The effect of cracks on deterioration depends on their origin [NCHRPS57 79]. Diagonal and initial random cracks are shallow and do not appear to be significant defects. Pattern cracking due to the use of reactive aggregates will eventually result in complete disintegration of the concrete so that the entire deck must be replaced [NCHRPS57 79]. Although this type of failure is obviously serious, it is not common and cannot be halted after improper aggregates have been used in the initial deck construction.

Cracking affects deck deterioration by providing ready access of chlorides, oxygen, and moisture to the reinforcing steel. Longitudinal cracks will thus hasten the corrosion of the intercepted transverse bars. Transverse cracking has even more severe deterioration consequences since the crack follows the length of the bar and resistance of the concrete to spalling is reduced [NCHRPS57 79].

A type of cracking deterioration which has been widely observed in Japan, but not in the United States, is water infiltration of penetrating cracks [Kato 84]. Initial pattern cracks due to drying shrinkage propagate through the entire deck thickness under service conditions. Water leaks through these full depth cracks and the crack surfaces are abraded against each other under traffic loads. This deck degradation due to water infiltration of cracks penetrating the full slab depth are observed on asphalt overlaid decks that have seldom been treated with deicing salts.

2.3 Spalling

The primary mechanism of concrete deck deterioration is corrosion of the reinforcing steel and the spalling that then occurs. Spalls weaken the deck locally, impair the riding surface, and expose the reinforcing to further corrosion. Neglected spalling can eventually lead to total deck failure.
2.3.1 Corrosion of steel in concrete

Spalling is mainly due to corrosion of the steel in the top rebar mat. Corrosion of steel in concrete is an electrochemical process involving a flow of electrical current coupled with chemical reactions [Verbeck 75]. It is a wet corrosion process, the metal reacts with an aqueous solution. Four elements are required for corrosion activity: an anode, a cathode, a conductor, and an electrolyte [ACI 222 85]. The anode is the positively charged electrode, toward which current flows, and where electrochemical oxidation takes place. Metal is converted from a non-ionic to an ionic state at the anode as electrons are released through oxidation. Corrosion products accumulate at the anode. Current flows from the negatively charged electrode, the cathode, as it consumes the excess electrons generated at the anode. Electrochemical reduction occurs at the cathode. The electrons flow through the conductor. The electrolyte is the conducting medium through which the ions move between the anode and the cathode.

For corrosion of steel in concrete, the iron of the reinforcing acts as both the anode and the cathode. The inhomogeneities of reinforced concrete allow electrical potential differences to develop between different locations on the reinforcing due to differences in moisture content, oxygen concentration, cracking, and residual steel stress [NCHRPS57 79]. A macrogalvanic corrosion cell is established along a rebar with anywhere from an inch to more than twenty feet separating the anode and the cathode. The reinforcing is the conductor, transmitting electrons. Moisture acts as the electrolyte. Moisture and oxygen are also required to support the chemical reactions of corrosion.

The chemical reactions of iron corrosion are summarized in Figure 2-1. Iron enters into solution at the anode, freeing electrons. The electrons are consumed at the cathode by the formation of hydroxyl ions, if oxygen and water are present.
Ferrous hydroxide (white rust) is deposited at the anode and converted to ferric hydroxide (red-brown rust). This transformation to higher oxides results in a fourfold volume increase [ACI 222 85].

2.3.2 Mechanics of spalling

The formation of spalls is initiated by the corrosion of the top mat of reinforcement. The corrosion process causes rust to build up at the anode. The iron oxide corrosion products occupy more volume than the parent iron. This volumetric expansion exerts bursting forces on the concrete surrounding the rebar. These tensile forces are much greater than the tensile strength of the concrete. The concrete fractures to relieve these stresses, forming a plane of delamination. The shape of the planes of delamination differ, depending primarily on the depth of cover, as shown in Figure 2-2 [Stark 71]. Traffic, freeze/thaw, and continuing corrosion causes further cracking so that a connected fracture plane grows around a section of the surface concrete. Eventually the surface concrete piece breaks free, leaving a spalled depression in the deck. Fig 2-3 [NCHRPS4 70] illustrates the formation of a spall.

2.3.3 Role of chlorides in the spalling process

Concrete has a naturally high pH level due to calcium hydroxide and alkalis in the cement, with a normal pH value of about 12.6 [Treadway 79]. In alkaline environments with a pH greater than 11.5 steel corrosion is inhibited by the formation of gamma ferric oxide which prevents further oxidation at the anode [Cady 83]. This halting of corrosion by the alkalinity of the concrete is called passivation. Sufficient quantities of soluble chloride ions can destroy passivation. A concentration of chloride contaminants in the concrete above a critical threshold initiates active corrosion if moisture and oxygen are also present. Since water and
Figure 2-1: Corrosion of Steel in Concrete
Figure 2-2: Planes of Delamination

- Inclined (Trench) Cracks
  - \( L \leq 2.54 \text{ cm (1 inch)} \)

- Horizontal Fracture Plane
  - \( L \geq 3.18 \text{ cm (1.25 inch)} \)
Figure 2-3: Formation of a Spall

Crack formed by shrinkage, resistance to subsidence, thermal stresses, thin cover.

This area subject to stress reversal. Tension exerted by corrosion and ice, compression exerted by traffic.

Insufficient Cover

Salt solution in crack accelerates corrosion

CONCRETE

Crusting promoted by excessive fines, high mix temperature, tardy curing, results in bleed water trapped under top surface.

Weakened by trapped bleed water.

Re-Steel

SLAB

Ice lenses can form in fracture.

Brine percolates thru high W/C Concrete.

Accumulated salt at base of crack acts as anode in galvanic cell.

Products of corrosion exert powerful force.

EVENTS OCCURRING DURING CONSTRUCTION

EVENTS OCCURRING AFTER CONSTRUCTION
oxygen are almost always present at the reinforcing, the initiating event for active corrosion is a critical chloride concentration at the level of the top rebars.

Chlorides reach the level of the steel either by infiltrating cracks or by diffusion through the cover. Of these two access paths, infiltration is much faster. If extensive transverse cracking exists, corrosion is initiated with the first season of salting. Since low cover exacerbates transverse cracking, increasing the cover will delay the onset of active corrosion by decreasing the extent of cracking.

Cracking is not required for the initiation of corrosion. Sufficient chlorides can diffuse through the cover to eventually exceed the critical threshold. The length of time that it takes chlorides applied to the deck surface to penetrate the cover is dependent on the concrete permeability and the cover thickness.

Factors affecting permeability are primarily [Verbeck 75] water/cement ratio and curing. Other factors include [NCHRPS57 79]: cement/aggregate ratio, aggregate grading, air-entrainment, consistency, and degree of consolidation. Decreasing the permeability of the cover can lead to decreased corrosion for a reason other than the reduction in chloride diffusion rate. Low permeability concrete has low porosity so that less water can be retained, which lowers the concrete conductivity, and hence the corrosion rate.

Increasing cover thickness has a more than linear benefit in delaying the time chlorides reach the steel. The time to critical contamination is an exponential function of cover depth. This relational form has been established empirically [Spellman 70]. It is also in accordance with the expected behavior of a diffusional process governed by Ficks Law [Cady 83] summarized as:

\[ \frac{dx}{dt} = D \left( \frac{\partial^2 C}{\partial x^2} \right) \]

where \( C \) = concentration
\[ x = \text{distance} \]
\[ t = \text{time}, \text{ and} \]
\[ D = \text{diffusion constant}. \]

The time to critical chloride content thus varies more strongly than linearly with the amount of cover. This can be seen from the fact that the rate of concentration change with time is equal to the rate of the rate of concentration change with distance form the surface. Another reason for the slowing in diffusion with increasing cover is the formation of calcium chloroaluminate, which binds some of the chloride into a nonsoluble form, hence reducing the concentration gradient which drives the diffusion process [Verbeck 75]. Figure 2-4 [ACI 222 85] illustrates both the normal diffusion of an electrolyte into a porous solid without the chemical reaction, and the reduced diffusion due to the chemical reaction.

Since chlorides diffuse through even low permeability concrete, a protective barrier that is impermeable to brine must be put in place to prevent long term corrosion of embedded reinforcing steel. This is the objective of installing the various sealed overlay and membrane systems.

### 2.4 Scaling

Scaling is the breakdown in the cement-paste matrix of the concrete, caused mainly by the expansion of freezing water in the void system of the cement paste. Flaking away of the surface mortar allows the aggregate to loosen. In severe scaling the cement matrix deteriorates completely and aggregate can be scooped out by hand [NCHRPS57 79]. Scaling begins at the surface of the deck and can progress through the entire deck thickness. This type of deterioration is usually seen in concrete that has insufficient air-content to provide durability under freeze-thaw cycling. This concrete degradation is a severe problem in New England. In some
Figure 2-4: Chloride gradient: with and without chemical reaction with cement
states, for example New Hampshire, this concrete breakdown is perceived by bridge maintenance engineers to be an even more widespread problem than delamination deterioration.

2.4.1 Void structure of hardened cement paste

Concrete is a composite material composed of aggregates bound in a matrix of hardened cement paste (HCP). Knowledge of the void structure of HCP is necessary to understand the mechanisms of freezing and thawing of concrete. Three types of voids exist in the HCP [Cordon 66, Illston 81]. The largest are entrained air voids, which measure from a few millimeters to a few microns in diameter, with an average size of about 0.002 inches. These air voids are generated by intentionally trapping air into the plastic concrete during mixing. For concrete that is to be exposed to freeze/thaw, about 5 to 6 percent by volume air is deliberately entrained in the concrete to improve durability. Capillary cavities with an average size of about 5000 angstroms are formed by unhydrated water in spaces not filled by the tobermite gel sheets of which the HCP is composed. The smallest voids are the gel pores. These pores within the gel sheets are about 15 to 20 angstroms in diameter and occupy 25 to 28 percent by volume of the HCP.

2.4.2 Mechanics of scaling

Scaling is caused by freeze/thaw action. Water expands about 9 percent in volume as it freezes. If a void is more than 91 percent saturated, freezing of the water will more than fill the void. Unless the excess water can move out of the void, high bursting pressures will be exerted on the HCP surrounding the void as the ice crystal grows. When moisture in concrete reaches or exceeds the critical saturation point of 91 percent, vulnerability to freezing damage is maximized.
Consideration must be given to the size of the various voids to determine the movement of water under dropping temperature. The gel pores are of such small size that the water contained in them, under high pressure due to surface tension and adsorption, has a freezing point below -108 degrees F (-78 degrees C). Since only a few tens of water molecules fit in a gel pore, this water becomes supercooled but not frozen. Capillary cavities are large enough to allow formation of ice crystals and freezing [Cordon 66]. Capillary action insures that water will fill the capillary cavities before filling the larger the air voids. For non-saturated air-entrained concrete, cavity water can escape to the empty air voids when the capillary cavities begin to freeze. If the air voids are filled with water or if the distance between voids is too great, cavity water cannot escape and the resulting hydraulic pressure fractures the HCP and scaling results.

The void spacing is the most important characteristic of the air void system for durability [NCHRPS4 70]. Concrete may have an adequate volume of entrained air and still have poor freeze/thaw durability. The air voids must be distributed in closely spaced pattern so that capillary water does not have to migrate far to relieve pressures. The spacing of air voids should not exceed 0.01 inches in order to obtain adequate protection for freeze/thaw [Cordon 66]. The air void system becomes ineffective as a pressure relief mechanism if the voids are full or nearly full of water.

Scaling occurs at the deck surface as the HCP fractures to relieve freezing stresses. The breakdown of the cement matrix then occurs at the next deeper layer in the next freeze cycle. In severe cases, scaling can proceed in this fashion until mortar integrity is lost for the full depth of the deck.
2.4.3 Role of chlorides in the scaling process

Chlorides exacerbate scaling by affecting osmotic pressure, degree of saturation, and near-surface temperature. When chlorides are dissolved in the capillary moisture, initiation of freezing in a cavity will concentrate more chloride in the not yet frozen fraction of the cavity moisture. This increase in chloride concentration will cause an osmotic pressure that will draw water from the unfrozen gel pores. This osmotic pressure combines with the dilatonic pressure of the ice formation and can fracture the near-surface HCP [NCHRPS4 70].

Dissolved salt lowers the vapor pressure of the moisture in the concrete. This drop in vapor pressure increases the saturation of the concrete, thus increasing the water in the air voids and decreasing the ability of the void to relieve pressures during freezing [NCHRPS57 79].

Chlorides also increase the degree of saturation of the near surface concrete by melting surface ice and snow, thus providing a water source. A near-surface temperature drop is also caused by the melting of surface ice due to the heat of fusion. This temperature drop contributes to freezing of the water within the near-surface HCP voids.

2.5 Overlays, debonding, and membrane failure

Deck overlays are of two major types: concrete overlays and bituminous overlays. Concrete overlays may be categorized as low-slump, polymer-modified, or internally sealed, depending on the means used to limit the permeability of the overlay. All types of concrete overlay act as integral parts of the concrete deck, provided that the base was properly prepared so that adequate bond was obtained between the concrete layers.
Bituminous overlays are usually placed on top of a waterproof membrane. Placing a bituminous pavement on a deck without a membrane is a questionable practice [NCHRPS4 70]. The paving must be designed with a void system to inhibit migration of asphalt to the surface under traffic and high temperature. This inherent porosity of bituminous paving allows brine to reach and remain on the deck surface. Membranes with bituminous overlays are the most common type of deck protective system in place in New England.

There are two kinds of membranes, those which are applied-in-place liquid materials and those which are preformed sheet systems. Applied-in-place membranes, also called built-up membranes, were used extensively in New England beginning in the mid sixties [NCHRPS4 70]. Sheet membranes have predominated in the region for projects taking place since the late seventies. The decks that are the most likely to be experiencing distress are those which have been in service longer. For New England, these older bridges are likely to have applied-in-place membranes. Figure 2-5 shows a typical protective systems of this type [NCHRPS4 70].

Both types of membranes may develop debonding of the overlay from the concrete. The different coefficients of expansion of asphalt and concrete make such bond failures likely. When this bonding defect occurs, a crack plane exists at the interface of the overlay and the deck. This debonding plane can be difficult to distinguish from the far more serious crack plane defect of a delamination within the concrete deck.

There are several characteristics of asphalt overlays that can exacerbate deck deterioration [NCHRPS57 79]. The overlay masks early stages of deck distress so that deck deterioration is not detected until it is far advanced. The pavement allows chlorides and water to reach and lie on top of the deck. This creates an environment
Figure 2-5: Applied-in-place Water Barrier Membrane
highly conducive to deck deterioration if the membrane fails to keep moisture from the deck. Experience in New England has shown that by the time a bridge needs to be repaved [5 to 15 years], the applied-in-place membrane is typically not functioning as a moisture barrier.

If there is no membrane, or if the membrane has failed, a bituminous overlay will increase both spalling and scaling distress. The pavement generally remains saturated year round. It acts like a brine filled sponge continually soaking the deck. This provides a reservoir of chlorides, moisture, and dissolved oxygen for the corrosion process, thus aggravating spalling. The soaked concrete becomes saturated. Air-entrained concrete is very susceptible to frost damage if it is fully or nearly saturated. Its high porosity means it contains a large amount of water for freezing, and hence high scale damage at each freeze/thaw cycle [NCHRPS57 79]. An additional adverse effect of bituminous overlays is that the dark asphalt surface absorbs solar radiation better than concrete and therefore increases the number of freeze-thaw cycles that the deck must withstand. Under these conditions, severe scaling damage can occurs.

2.6 Summary of parameters affecting deterioration

2.6.1 Factors affecting spalling

Figure 2-6 summarizes the relationships among the various factors that affect spalling. A spall forms when a concrete piece, surrounded by a failure plane which has grown from a delamination, breaks free under traffic and/or freezing forces. The effects of the spalling forces are secondary since they only remove previously fractured segments. The delamination forms due to corrosion. The steel corrodes in the presence of chloride contamination, water, and oxygen. Water and
oxygen will usually be present in sufficient quantities due to infiltration through cracks or diffusion through the cover. The rate of corrosion can be increased if the concrete has a high electrical conductivity. This will be the case if the concrete is porous and the climate is not extremely dry. Concrete porosity is linked to concrete permeability.

Chloride content at the level of the rebars can be above the depassivation threshold due to chlorides in the fresh concrete or contamination of the hardened concrete by surface applied chlorides. Excessive chlorides in the aggregate, use of seawater in the mix, and addition of accelerators containing chlorides are sources of chloride contamination in fresh concrete. Hardened concrete in a marine environment absorbs salt from splash and spray. Deicing salts are the predominant source of chloride contaminants. A deck will be subjected to more total salt applications due to its age and the severity of its climate.

The salt infuses into the deck by rapidly flowing along cracks or by slowly diffusing through the cover. Transverse and longitudinal cracks provide access from the surface to the bars. Low cover, high water/cement ratio, and poor placement practices are the main factors associated with these crack types. There is conflicting evidence as to whether superstructure type, span continuity, span length, and deck form type can be reliably correlated with degree of spalling [Balduman 83]. Rapid diffusion results from low cover and high permeability. The major parameters affecting concrete permeability are high water/cement ratio, and poor placement and curing practices.

In addition to these factors, spalling will be more severe for a deck with a bituminous overlay without an intact waterproof barrier. Brine will be trapped in the paving which will lead to more moisture in the deck and will prevent salt from being flushed off by rain.
Figure 2-6: Factors Affecting Spalling
2.6.2 Factors affecting scaling

The various factors that affect scaling are illustrated in Figure 2-7. Freeze/thaw cycling repeatedly exerts osmotic and dilatoric overpressures on the HCP matrix of the concrete. The osmotic pressures are increased if chlorides are present. The expansive forces from ice dilation will fracture the mortar if air voids are not available for pressure relief, either because of inadequate air entrainment or excessive saturation. The saturation of the concrete is increased if chlorides are present or if the road profile allows ponding of water on the deck surface. The severity of the scaling damage in each freeze will be accentuated if a larger amount of water per unit volume of mortar is available for freezing. Higher concrete porosity will allow more water to be absorbed. Porosity is increased by high water/cement ratios, improper finishing, and inadequate curing. Scaling progresses as freezing is repeated. The total number of freeze/thaw cycles that a deck has been subjected to depends on the age of the bridge and the number of freeze/thaw cycles per year, determined primarily by the climate but also affected by salting.

In addition to these factors, a bituminous overlay without a functional membrane will increase scaling damage. The bituminous pavement tend to saturate the deck. The presence of paving also increases the number of freeze/thaw cycles.
Figure 2-7: Factors Affecting Scaling
Chapter 3

Current practice for deck assessment

3.1 Frequency and scope of inspections

The role of bridge inspections is to determine maintenance priorities, replacement priorities, structural capacity, and cost of maintenance. Inspections provide information used to quantify the labor, equipment, materials, and funds necessary to maintain the integrity of the structure [AASHTO 76a].

Inspections of bridge decks differ in scope and frequency depending on the purpose of the investigation. The general condition of all bridges within a state's jurisdiction is determined by routine inspections on a regular basis so that a reasonably accurate inventory can be maintained. This inventory data can be used to plan normal maintenance and to identify those bridges which may require more extensive attention. In addition to these routine inventory inspections, interim inspections are conducted on those bridges that are weight limited, structurally deficient, or special need [AASHTO 76a]. Weight limited structures that are not capable of carrying the state's legal load limit are inspected annually. Structurally deficient bridges are inspected as often as judged necessary to assure the safety of the public and the integrity of the structure. Special inspections include assessment of bridges damaged by storm or accidents, routing of overweight vehicles, and gathering information for possible reconstruction or replacement. The two major categories of condition surveys are routine inventory inspections and preconstruction investigations.
3.1.1 Inventory inspections

Inspection of every highway bridge, more than twenty feet long, on a public road in the United States is required at regular intervals, not to exceed a maximum of two years, in accordance with the 1968 Federal-Aid Highway Act as expanded by the 1978 Surface Assistance Act. The event that prompted Congress to establish a National Bridge Inspection Standard was the catastrophic failure of the Silver Bridge in 1967. This bridge, spanning the Ohio River from Point Pleasant, West Virginia to Kanauga, Ohio, collapsed without warning, killing 46 people. The failure was caused by the brittle fracture of one of the suspension eyebars [Fisher 84].

The depth and frequency of bridge inventory inspections may be varied by the states, subject to the two year maximum time interval. Factors such as age, traffic characteristics, state of maintenance, and known deficiencies determine the appropriate inspection level. States may conduct the mandated biennial surveys as part of general maintenance of the highway in the area of the bridge. The purpose of these inspections is not only to identify existing defects, but also to anticipate problems. This allows the data to be useful for undertaking not only corrective, but also preventive maintenance [FHWA83 83].

The National Bridge Inventory specifies 90 attributes that are to be entered for each bridge. Of these attributes, 16 are condition items, of which one is the appraisal of the deck condition. Concrete decks are checked for type and extent of deterioration, primarily cracking, spalling, and scaling. Since an asphalt wearing surface can obscure damage until it is far advanced, small sections of the overlay may be removed so some patches of the deck surface can be examined. Visual inspection of the underside of the deck slab is usually performed, whether or not an overlay is in place. Visual observation of the deck from above and below by an
experienced inspector is the method that is relied on most heavily to determine a deck rating for the biennial inventories.

3.1.2 Preconstruction surveys

An in-depth field survey is usually undertaken for any bridge deck that has been identified as a candidate for repair of replacement. Bridges may become candidates based on the results of routine condition surveys, or when work is programmed for the road segment of which the bridge is a part. When the condition survey indicates that the deck may not be adequate to carry traffic loads, an in-depth investigation to determine structural condition is necessary for safety's sake. When major work, such as repaving, is planned for the route on which the bridge lies, the inadequacies of the deck must be established so that the appropriate deck construction can be scheduled.

For decks in advanced stages of deterioration, an in-depth survey may not be needed. Visual methods may be sufficient to determine and document that complete deck replacement is required. For decks with less extreme signs of distress, an in-depth survey is usually required to determine the type and extent of damage.

The Federal Highway Administration recommends that a detailed field appraisal should consider the following items, as appropriate for a particular deck [AASHTO 76b]:

- Delamination detection to determine extent of internal fractures
- Half-cell corrosion tests to determine the extent of reinforcing corrosion by measuring electrical potentials
- Chloride content chemical analysis tests to determine the extent of chloride contamination at the level of top reinforcement.

This information is used to decide if repair or replacement is required. If deck
repairs are planned, the results of the condition survey are used to select appropriate repair methods, prepare construction documents, and estimate bid quantities.

3.2 Current inspection methods

The various well-established methods of data collection for deck condition appraisal may be roughly ordered from most to least frequently used as: visual inspection, delamination detection by sounding, measurement of chloride content, core drilling and testing, measurement of corrosion potentials, cover measurement by pachometer surveys, and electrical resistance testing of membrane integrity.

3.2.1 Visual inspection

The first step in any condition survey is usually a visual inspection. All visible defects on both top and bottom deck surfaces are categorized and recorded. The size, location, extent, and depth of spalling and scaling are noted. The location, length, width, and orientation of cracks are observed, and if possible, the cause of cracking is determined. Visual inspection is complicated if an asphalt overlay hides the deck. Overlay cracking and wet spots may be indicative of underlying deck distress. If the underside of the deck is accessible, it is examined for signs of leakage and deterioration such as: efflorescence, wet spots, cracking, and rust stains [FHWA83 83].

3.2.2 Detection of delaminations by sound

Delaminations mechanically separate the upper layer of the concrete from the bulk of the deck. When a delaminated area of the deck is struck with a tool, this separation makes the noise sound dull and hollow in comparison to the more highly
pitched ringing noise of an intact portion of deck. Hammers, rods and chains are used to strike the deck. Chains are most common, so this type of test is sometimes simply called a "chain drag" test. This traditional method of identifying delaminations has been found to be effective for inspection of exposed concrete decks [Manning 82].

Efforts have been made to automate this delamination detection process in order to increase its speed, decrease its dependence on operator skill, and decrease the tedium of recording data. A commercial product, the Delampect [NCHRPS57 79], consists of a tapping device, a sonic detector, and a pen recorder, all mounted on a small hand pushed cart. Since the Delampect only tests a 150mm (6") wide strip, a series of passes in a grid pattern are necessary to obtain adequate deck coverage and small isolated delaminations would be expected to be missed at the 1.5 m (4') grid spacing commonly used [Manning 82]. This device is not as accurate as hand methods.

3.2.3 Chloride content chemical testing

Chemical analysis of a sample of concrete from the level of the top reinforcing mat can be used to determine whether the deck is contaminated with chlorides. The number of samples that must be taken to obtain a data set representative of the overall deck condition is dependent on that deck's variation in chloride content. Six samples are commonly recommended as a minimum set. Different approaches are taken to locate samples. Samples can be randomly located, located in an attempt to maximize variation, or located in areas of ambiguous results from other test methods. Samples may be taken in either cored or pulverized form for laboratory analysis. A rapid in situ method has also been developed, which is less accurate than laboratory methods, but has the advantages of being quick and causing minimal, easily repaired deck damage [NCHRPS57 79].
3.2.4 Core drilling and testing

Another sampling technique that is commonly used to supplement nondestructive methods is core drilling. Since this is a discrete sample method, questions of sample number and location again arise. On exposed decks one core is usually taken for every 2000 square feet of deck, with a minimum of three cores. For an asphalt overlaid deck this amount would be increased three or four fold. In this case, sections of the overlay are usually removed in rectangular patches to allow visual inspection of the deck at sample locations. Cores are usually taken in these stripped areas. The cores are examined in the laboratory to determine: visual signs of deterioration, aggregate and cement paste condition, air voids, density, strength, and chloride content.

3.2.5 Corrosion potentials

As the reinforcing rusts, current flows within the macrogalvanic cell from the anode to the cathode. This current flow creates a potential difference between the anodic half cell and the cathodic half cell. A voltmeter can be used to measure this corrosion potential against a reference potential. The standard version of this test uses a copper/copper-sulfate (CSE) cell to provide the reference voltage. The standard equipment and procedure are specified by an American National Standard [ASTMC876 85]. A reading of less than -0.20 volts CSE is interpreted as greater than 90% probability of no active corrosion in the area; the range from -0.20 to -0.35 volts CSE is uncertain; and a reading of more than -0.35 volts CSE is interpreted as greater than 90% probability of ongoing corrosion. This method detects only the likelihood of corrosion and does not detect the rate of corrosion. When used on asphalt-covered decks, it is desirable, if no membrane is present, and essential if a membrane is present, to drill through the paving to ensure contact with the concrete [Manning 82].
3.2.6 Pachometer surveys of cover depth

The thickness of the concrete cover over the top layer of reinforcement can be measured by a magnetic device called a pachometer. The pachometer generates a magnetic field between two poles on a probe. This field is distorted by ferromagnetic materials, such as steel. The pachometer detects the magnitude of this field distortion, which is proportional to the size of the bar and its distance from the probe. If the size of the bar is known from construction drawings, the cover thickness can be determined directly, otherwise cores can be used to determine bar size at several points. Cover measurements are used to establish the depth of the top steel for taking chloride samples, determine if low cover is the cause of observed deterioration, and locate areas with insufficient cover to allow scarifying [Park 80].

3.2.7 Electrical resistance of membranes

Most membrane materials are not electrically conductive. The electrical resistance of a membrane can be assumed to be indicative of its permeability since the openings that allow moisture to pass will also allow current flow. The resistance of the membrane can be measured by connection of an ohmmeter to the top of the membrane and the top layer of reinforcing. The standard equipment and procedure to perform this test are specified by an American National Standard [ASTMD3633 85]. Since epoxy is an insulator, decks with epoxy coated top mats cannot be tested with this method. In addition, pavement porosity and moisture variability can cause problems in obtaining meaningful readings. If the readings are taken from the paving surface instead of stripping the overlay at the test points, the readings are not very reliable. This test method is susceptible to error. There is no general agreement on interpretation of the data so the evaluation of the results is subjective. Electrical resistivity tests are unable to discriminate between
membranes with a few pin holes, which offer good deck protection, and those with large punctures, which fail to offer protection [Manning 86].

3.3 Current practice in five New England states

3.3.1 Inspection types

All of the participating states perform the federally mandated biennial inspections that are part of the National Bridge Inventory program, so no further description of inventory inspections is necessary. There is some variation among the states in how bridge work is programmed and thus in how bridges are selected for reconstruction surveys.

Maine has an orderly and comprehensive screening process for bridge project selection. The selection of which bridges need an in-depth survey begins with the Bridge Inspection Program. A deficient bridge list is developed as part of this inspection program which is performed by the Maintenance Department. The Posting Committee then reviews this list and, considering the budget constraints, selects projects for bridge repair and bridge replacement. The project list is then sent to the Planning Division which puts together a package for the Funding Committee which decides on the actual list of funded projects. The funded list then goes to the Bridge Design Section for preliminary engineering. The first step of preliminary engineering is to conduct field tests. The Materials and Research Division does the field testing in response to a request from the Bridge Design Section.

In Massachusetts, the current practice for assessing deck condition is decentralized. There are NBI inspection teams for periodic inventory surveys but there is little extensive condition survey activity beyond that work. The primary
source of information for project selection is the district engineers. The priorities are set by the districts, and which projects are done is influenced by district talent. In addition to district initiated requests, limited use is made of National Bridge Inventory data for identifying candidate bridges on the interstate system.

New Hampshire's Materials and Research Division receives requests for bridge testing primarily from Bridge Design when a bridge is part of a highway paving project, and secondarily from Bridge Maintenance when a problem is reported that seems to warrant further investigation. Materials and Research contracts out the testing and evaluates the results to estimate repair quantities and recommend a repair/replace strategy.

Rhode Island selects candidate bridges for repair or replacement primarily on the basis of what bridges lie within the bounds of a planned highway repaving project. Until recently testing was done in-house by the Materials Department, but due to a large increase in work load, the work has begun to be contracted out to consultants.

In Vermont, those bridges that lie on a highway/road segment which is scheduled for paving and those bridges scheduled for the federal 4R program are chosen as the initial pool of decks which will be subjected to a preliminary survey. Priorities are thus set by paving segments, not by considering the bridges as separate entities. In the autumn, the Structures Division submits this list of bridges to Materials and Research which then schedules a preliminary survey to be performed on each bridge during the winter.
3.3.2 Inspection methods

The current methods of assessing deck condition which are used in one or more of the five states are:

- Knowledge of age and in-place protective system. This information is usually available from the states' bridge inventory databases, supplementing information recorded for the NBI. Knowing the history of a deck can provide valuable information when evaluating the test results and determining the optimum repair or replacement strategy.

- Visual inspection from above. This method, when done by an experienced bridge inspector, is the single most reliable and informative source of information for condition assessment.

- Visual inspection from below. Extensive cracking and efflorescence are definitive indicators of deck deterioration, but these signs show up too late to be of much practical use.

- Visual inspection of sample areas which have been stripped of their bituminous overlays.

- "Sounding" techniques, such as chain drag, to delineate delamination on exposed decks.

- Determination of chloride content at the level of the top reinforcing mat.

- Core sampling and examination.

- Determination of galvanic corrosion cell electrical potentials (corrosion potential, half-cell test).

- For decks with a bituminous overlay, sample areas are stripped so the deck below may be examined for visual signs of distress and also for delamination detection by sounding.

Table 3-I shows what test methods are used in each state.
Table 3-I: Current Inspection Methods Used in Five New England States

<table>
<thead>
<tr>
<th>Method</th>
<th>Maine</th>
<th>Massachusetts</th>
<th>New Hampshire</th>
<th>Rhode Island</th>
<th>Vermont</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strip, Sample Patches,</td>
<td>Paved Decks</td>
<td>None</td>
<td>Limited Only</td>
<td>None</td>
<td>Limited</td>
</tr>
<tr>
<td>Overlay</td>
<td></td>
<td>None</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Corrosion Potential</td>
<td>None</td>
<td>None</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Concrete Cure Samples</td>
<td>Yes</td>
<td>Limited Only</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Chloride Content Samples</td>
<td>Yes</td>
<td>Limited Only</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Chain Drag Delamination</td>
<td>Bare Decks Only</td>
<td>Yes</td>
<td>Bare Decks Only</td>
<td>Yes</td>
<td>Limited</td>
</tr>
<tr>
<td>Detection</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Visual, Top and Bottom</td>
<td>Yes</td>
<td>None</td>
<td>None</td>
<td>None</td>
<td>Yes</td>
</tr>
<tr>
<td>Inventory Survey Data</td>
<td>Yes</td>
<td>None</td>
<td>None</td>
<td>Limited</td>
<td></td>
</tr>
</tbody>
</table>

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3.4 Critique of current deck assessment practices

3.4.1 Inspection types

The inspections that are performed in all states as part of the National Bridge Inventory have problems relating to data objectivity, accuracy, and utilization. Since the deck ratings are based primarily on visual inspection, they are subjective and have the potential for variation and inaccuracy depending on the judgment and perception of the inspector.

In addition to variability and inaccuracy of the rating determination, there are problems with data quality within the inventory data bases. Errors can be introduced during coding, duplications and omissions may be made, and revisions may not be made in a timely fashion. An informal review of Massachusetts inventory information indicated that the error rate may be nearly 50 percent [Seymour 85]. A more detailed examination of a sample set of records from the national data base supports this conclusion of erroneous data. In this second study, 30 percent of the records examined were rejected due to implausible codings [Busa 84].

As a final point, the potential usefulness of the National Bridge Inventory data is frequently left unrealized. This lack of data application is true in New England, as can be seen in Table 3-II [Seymour 85].

The differences in procedures among the various states make it difficult to generalize about the way bridges are programmed for work. Massachusetts currently addresses bridge problems in a decentralized manner, as each individual case is brought to the attention of the central office by the separate district offices. The state is making efforts to establish more uniform procedures for programming bridge work. For Maine, which has a formal screening process, the current initial
**Table 3-II:** Use of NBI Data in New England and Other States

<table>
<thead>
<tr>
<th>STATE</th>
<th>ROUTINE MAINT</th>
<th>MAJOR MAINT</th>
<th>BRIDGE CAPAC</th>
<th>OVERWT ROUTE</th>
<th>NEEDS FORE-MAINT</th>
<th>PROGRAM</th>
<th>PROGRAM ESTIM CLEAR CAST PROGRAM</th>
</tr>
</thead>
<tbody>
<tr>
<td>ME</td>
<td>none</td>
<td>none</td>
<td>limited</td>
<td>limited</td>
<td>limited</td>
<td>none</td>
<td>none</td>
</tr>
<tr>
<td>NH</td>
<td>none</td>
<td>none</td>
<td>none</td>
<td>limited</td>
<td>none</td>
<td>none</td>
<td>none</td>
</tr>
<tr>
<td>VT</td>
<td>none</td>
<td>none</td>
<td>limited</td>
<td>limited</td>
<td>none</td>
<td>limited</td>
<td>limited</td>
</tr>
<tr>
<td>MA</td>
<td>none</td>
<td>limited</td>
<td>none</td>
<td>none</td>
<td>limited</td>
<td>none</td>
<td>limited</td>
</tr>
<tr>
<td>NY</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>PA</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>NM</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
</tbody>
</table>
identification of deficient bridges is based on the NBI data collection procedure and
the comments of maintenance personnel who are familiar with the appearance of
the bridge over a period of time. A better method is desired, one which will provide
a uniform and consistent method of selecting deficient bridges in a more objective
way. If an accurate, objective testing method were available to detect early deck
deterioration, the deck assessment could be combined with the pavement
management program. This approach would allow a single inspection program to
address both bridge deck and pavement surface evaluation. This would be
philosophically consistent with the approach taken by New Hampshire, Rhode
Island, and Vermont of programming deck work by road segment, not by considering
decks independently from the road of which they are a part.

3.4.2 Inspection methods

Visual inspection, when done by an experienced bridge inspector, has been
found by the participating states to be the single most reliable and informative
source of information for condition assessment. The reliability of visual inspections
depend on the perceptiveness and judgment of the inspector. However, there are not
always enough experienced inspectors to do the job. The supply of trained inspectors
varies with location. Vermont does not perceive a current problem in finding
enough qualified inspectors. On the other hand, Massachusetts has difficulty
finding a sufficient number, and has not been able to overcome this shortage with
training programs, since the turnover of inspection personnel is high.

Visual inspection, although an extremely useful method for bare decks, is a
problematic method when applied to asphalt-covered decks, even when performed by
experienced inspectors. The condition of the bituminous overlay may or may not be
indicative of the condition of the underlying concrete deck. In cases where an
effective membrane is in place, the overlay may be in poor shape when the deck is in good condition. Conversely, when no membrane or an ineffective membrane is present, the deck may be severely deteriorated when the overlay is in good condition [NCHRPS118 85]. The top visual inspection is not very informative, and the bottom visual inspection can only identify areas of extreme deterioration. If the underside of the deck is not accessible for inspection, or is hidden by stay-in-place forms, and an overlay is present, visual inspection is not of much use.

Sounding techniques, such as chain drag, accurately detect delamination on bare decks. This approach is inexpensive and not weather dependent but is tedious, dependent on the operator's skill, and time consuming. It is difficult to use on a bridge only partly closed to traffic, due to interference from traffic noise.

The experience in the region has been that sounding methods cannot differentiate between debonding and delamination. In addition, chain drag identifies only a small fraction of the delaminations on asphalt-covered decks [Manning 82]. The automated Delamtect equipment has been judged to be insufficiently accurate by the New England states, even for use on exposed decks. Accuracy of this instrument degrades substantially when it is used on asphalt-covered decks [Manning 83]. For these reasons, manual and automated sounding techniques are not promising for use on asphalt-covered decks.

Coring and chloride ion content tests may give results that are not representative of the overall decks, since a limited number of samples are taken. New Hampshire pointed out that the sampling methods of coring and chloride content can give extremely misleading results if the samples are taken from a part of the deck that the person collecting samples thinks are typical locations which are actually non-representative parts of the deck, for example over beams. In addition, the significance of a particular chloride content is ambiguous since it is not total
chloride but only free chloride which is available to destroy passivation. Since the correlation between free and combined chloride is not well defined, inferences based on the measured total chloride may be misleading.

Different states rely to differing degrees on the chloride ion content test. Vermont has found the results vary widely within a small area so that obtaining representative results is difficult. Other states, for example Maine, although not truly satisfied with the reliability of the results, use this test for decision making because their experience with alternative methods has been even less satisfactory.

Vermont has found the corrosion potential test, when used in a grid pattern, to be a very useful indicator of corrosion activity. Other states have had poor success with this method. It has the disadvantages of requiring lane closures which are unacceptable for the dense urban areas of Massachusetts and Rhode Island. In addition, for bridges which have a membrane, it is necessary to drill through the asphalt and puncture the membrane at the grid points, which would be detrimental if the membrane was previously intact. If the deck is paved, large amounts of moisture in the paving can conduct electricity over a broad area so the area of corrosion activity may not be identified. The test cannot be performed when the deck is frozen and it is recommended that both deck and ambient temperature be above 50 degrees F.

Pachometer surveys of cover depth do not provide direct information about deterioration, but instead identify areas of low cover which are likely to experience distress. If an overlay is in place, the pachometer identifies the distance from probe to bar through both overlay and cover, not the desired measure of concrete cover. Pachometers are used by several states to monitor deck cover during construction, but are rarely used for condition assessment.
Although resistivity tests have been tried by several states, the results were not found to correlate with the area of deterioration and this method is not in active use in the region.

3.5 Summary of capabilities and limitations of current methods

The testing methods currently in use have the following strengths:

- These methods permit the application of judgment and experience

- The techniques are well established and standardized.

- With the exception of chloride content and potential tests, methods measure the material properties of interest in a fairly direct manner.

- For exposed decks where lane closures are feasible, experience in the participating states has been that current methods can assess deck condition satisfactorily to decide on repair or replacement, but not to predict future condition.

The testing methods currently in use have the following weaknesses:

- These methods require the application of judgment and experience.

- The techniques are labor intensive and time consuming.

- The limited sampling is often unrepresentative.

- Lane closures are required for most methods.

- For asphalt-covered decks, these methods are not sufficiently reliable.

An interesting point is that application of judgment and experience is listed as both an advantage and a disadvantage. Condition assessment in practice is part art and part science. It is advantageous to make the most use possible of available inspection expertise. The fact that methods like visual inspection require a high degree experience in order to be reliable and complete is an obvious weakness.
The major limitations of all current methods are that they are slow, labor intensive, and require lane closures. Consequently, they can only be applied to a limited number of decks, usually those in the worst condition. Visual inspection of sample areas which have been stripped of their bituminous overlays; petrographic examination of cored specimens; chloride content; and potential tests all share the drawback of either depending on a small number of samples and hence risk being non-representative of the overall deck or requiring an extensive grid of test points and hence become time consuming and labor intensive. In addition, the lane closures required for these tests are unacceptable in urban areas.

In summary, in spite of the effort to systematically apply traditional methods, it is very difficult to assess the condition of an overlaid deck with the accuracy needed for maintenance management subject to tight funding constraints.
Chapter 4

Non-traditional technologies for deck assessment

4.1 Desiderata for new methods

The need for improved non-destructive testing methods for bridge deck evaluation has been clearly established. A wide variety of testing methods are available for consideration [Bungey 83Bungey2]. In order to identify those methods that have the most promise for meeting the needs of the New England states, the desired characteristics of a test method need to be made explicit. The performance of candidate methods can then be compared against the desired criteria so that the best methods may be selected as the focus of the research program.

A practical testing system should have the following characteristics:

• Nondestructive, so the condition of the in-service deck is not degraded and so the test may be repeated at a later time.

• Rapid, preferably noncontact, so the extent of deck surveyed may be large and so traffic is not constricted.

• Access independent, so testing is done only from on top of the deck.

• Reliable and accurate, to obtain information adequate for the decision process.

• Economical, to ensure cost-effectiveness and allow broad coverage.

• Flexible, so it may be applied to general deck configurations, including overlaid decks where the inadequacies of traditional methods are greatest.

• Weather tolerant, so testing may be performed under a range of environmental conditions.
Rugged, so the equipment is suited to field use.

Objective interpretation, so results are repeatable, quantifiable, and not dependent on the availability of skilled operators.

The selected techniques must not only have the correct capabilities to perform the deck assessment task, but must also be at the right level of technological maturity. The time and budget constraints for this research program rule out the development of new types of test equipment. The concept is instead to select promising existing equipment and develop applications and procedures to produce the condition data needed to make bridge management decisions.

4.2 Overview of Testing Methods

All testing methods used to assess bridge deck condition are based on the measurement of one or more parameters associated with some form of deck deterioration. For example, the measurement of electric potential is motivated by the fact that the voltage is one of the driving forces behind reinforcing corrosion. A test method measures one or more parameters associated with some form of deck deterioration so that the bridge deck condition can be inferred. For concrete bridge decks, the major items of interest are active corrosion, scaling, and delamination, and the associated parameters include: cracking, scaling, concrete permeability, chloride ion concentrations, concrete cover, concrete density, air-content, presence of oxygen and moisture at the reinforcing, electric potential, and overlay debonding.

4.2.1 Nondestructive testing methodology

The basic mechanism underlying all non-destructive testing techniques is that a source signal reacts with a physical system to generate an output signal which can be processed to determine the state of the physical system. To detect a
defect in the physical system, the presence of the defect must generate a different output signal from the signal returned by sound material. This difference must be observed and processed in a way that allows the presence of the defect to be inferred from the character of the output signal. The issues of instrumentation, data acquisition, data analysis, and data interpretation all play a part in nondestructive testing and evaluation.

4.2.2 Classification of testing methods by energy source

Nondestructive testing techniques are based on a fairly small number of scientific principles. The major areas can be categorized according to the form of energy of the input signal as: mechanical stress, electromagnetic, electric, magnetic, nuclear, and gravimetric.

Mechanical stress methods are those based on the propagation of elastic waves. Stress waves require a mass medium for propagation, since they are transmitted by direct molecular contact. The elastic properties of the medium control the velocity of stress waves through it [NCHRPS118 85]. Stress methods detect flaws based on the difference in stress wave propagation through sound material versus through deteriorated material. Since sound and deteriorated regions have different elastic properties they also have different stress wave propagation properties. A mismatch in wave propagation properties thus exists at the interface between the two types of material. This mismatched interface causes partial wave reflection. Detection and interpretation of the reflections can be used to infer the location and type of the reflecting interface. Mechanical methods have the advantage that they are a relatively direct means of determining mechanical properties, since a mechanical signal is used. One of their major disadvantages is that contact is required for both excitation and measurement. Mechanical stress
methods include: seismic and microseismic waves, sonics and ultrasonics, and
acoustic emission. Rebound and penetration methods are related to stress methods,
although they measure surface hardness and not bulk medium elastic
characteristics.

Electromagnetic radiation travels through vacuum as well as dielectric
media. The broad category of electromagnetic radiation may be further divided into:
gamma and x-rays; ultraviolet, visible, and infrared light; microwave (radar); and
short and long wave radio. The preceding lists the electromagnetic spectrum from
shortest to longest wavelength, which is the same as highest to lowest frequency.
Due to their greater energy levels, it is the shorter wavelengths that are applied to
nondestructive testing. Microwave is the longest wavelength in general use for
testing [NCHRPS118 85]. Electromagnetic methods have the advantage that they
usually noncontact and rapid. They have the disadvantage of being an indirect
method of testing mechanical properties. Electromagnetic methods include: methods
based on x and gamma ray sources, optical tests, thermal techniques, and radar.

Electric methods include resistivity/conductivity, potential, and impedance
tests. They can be used to find the voltage available for driving corrosion.
Resistivity measurements can be used to infer integrity of membranes and sealers.
Direct current polarisation resistivity measures could theoretically indicate the rate
of active corrosion [Fidjestol 80], although impedance measures are more promising
for development of practical corrosion rate field testing [John 81].

Magnetic methods can be used to detect ferromagnetic materials. The
presence of steel alters the field of an electromagnet. This principle can be used to
detect the location and size of reinforcing bars and hence can be used to detect
concrete cover.
Several nuclear methods have recently begun to be used for nondestructive field testing of concrete, though they are still in the exploratory stage [Bungey 82]. Nuclear magnetic resonance can be used to measure moisture content [NCHRPS118 85]. Nuclear methods can also measure chloride content, as can neutron moisture gauges [NCHRPS118 85].

Gravimetric methods are not applicable to testing the condition of bridge decks.

4.3 Summary of methods considered

4.3.1 Rebound and penetration tests

Rebound and penetration methods use a hardness measure to indirectly predict concrete strength. Several existing tests are: indenter tests, Schmidt hammer, and Windsor probe. All these methods required exposed concrete for their use and none are reliable for doing more than identifying anomalous areas without calibration for each deck by some other test [NCHRPS118 85]. These types of test method were not further investigated since all methods based on the indirect inference of concrete strength from surface hardness would have these drawbacks and therefore be of marginal interest for deck assessment.

4.3.2 Sonics and ultrasonics

Sonic and ultrasonic techniques use sound waves. The presence of cracks, reinforcement, voids, microcracks, and moisture affect the transmission of sound in concrete so these methods have been investigated for applicability to deck assessment. Sonic and ultrasonic pulse-velocity methods use a velocity measure of mechanical stress waves to infer concrete strength. These methods are indirect (the
velocity is related to the elastic modulus which is related to the void content which is related to the compressive strength) since the material property of interest is only indirectly measured by the test energy. These methods have the undesirable characteristic of requiring mechanical contact for both source signal and measurement.

Sonic reflection techniques are based on monitoring the audible sound produced by striking the deck. The presence of delaminations is detected by a change in frequency of the sound of an impact on the deck. The traditional chain drag test and the automated Delamtect device are sonic techniques that have been investigated for use on asphalt-covered decks. Sonic reflection has been found to have a very low accuracy for this application [Manning 85].

Ultrasonic transmission involves introducing high frequency sound waves into the deck and measuring the time of wave travel. The speed of sound in a homogeneous material is a function of the material’s density and elastic constants. In concrete, the heterogeneous composition, porosity, and moisture content also affect the speed of sound. The reflections of the sound waves at interfaces can be used to detect discontinuities. Ultrasonic transmission has certain limitations which render it impractical for concrete deck surveys [Manning 82]. If direct transmission of pulses is used, the source and receiver must be located on opposite sides of the deck with a known path length between the transducers. The path length through a deck is not known with sufficient accuracy to detect signal changes due to variation in concrete quality. Indirect transmission, with both transducers located on top of the deck, has even less path length certainty and hence cannot produce meaningful results.
4.3.3 Seismic and microseismic waves

Seismic waves are capable of resolving only large discontinuities and therefore are not applicable to deck assessment. Their main application area is to geophysical investigations. Microseismic refraction uses spaced geophones to time the travel of a shock wave through the test material. When investigated for bridge decks [Manning 83], microseismic refraction detected defects, but could not define their extent. The method is slow and difficult to interpret.

4.3.4 Acoustic emissions

Acoustic emissions are the low frequency (often in the 50 to 100 KHz range) sounds that most materials emit as they are deformed. As a material is loaded, kinetic energy is released by localized yielding, crushing, or microcracking. This energy release produces small amplitude stress waves which propagate through the material [Bungey 82]. Although acoustic emission techniques have been cited as having potential for bridge deck evaluation [Cantor 78], the applications to concrete are not fully developed and are still regarded as essentially laboratory methods. Serious technical difficulties need to be surmounted before acoustic emission can be used as a field testing method [NCHRPS118 85].

4.3.5 X-ray and gamma ray sources

X-rays and gamma rays are distinguished only by their origin. X-rays are produced by extra-nuclear atomic processes while gamma rays are usually nuclear in origin [NCHRPS118 85]. Local density gauges using gamma rays and density mapping techniques using x or gamma rays could detect deck defects [Joyce 84]. A Compton scatter gauge, measuring the collimated back scatter of a collimated gamma source, is sensitive to local density variations, but has a limited field of view and requires long time exposures for each reading.
Computerized tomography is a density mapping technique that offers the attractive concept of developing a nondestructive cross-sectional view of the deck. For a stationary object, tomography obtains a clear image of one plane, while blurring all other planes, by moving the radiographic source and film. Commercial equipment is not yet available and the practical feasibility of this method has not been established [Joyce 84]. More conventional techniques based on x-rays are not sensitive to fracture planes perpendicular to the direction of radiation [Manning 82] and require access to both sides of the test sample.

Methods based on x-ray or gamma ray sources do not appear to be easily applicable to deck assessment and only show significant promise as an inspection technique for prestressed concrete structures [NCHRPS118 85].

4.3.6 Optical methods

Optical techniques have the advantage of being rapid and high resolution. Direct visual observation of top and bottom slab surfaces is part of current inspection practice. Laser and other optical methods do not appear to have promise for detection of defects in asphalt overlaid decks, due to their disadvantages of being only a surface test and requiring a line of sight.

4.3.7 Electric methods

Resistivity/conductivity measurements are used to infer the integrity of membranes and sealers. Potential tests indicate the likelihood of active corrosion, but do not provide information on the corrosion rate. Resistivity and potential tests are elements of current practice for deck assessment.

Alternating current impedance tests can be used to estimate corrosion rates [John 81]. Since impedance methods are transient in nature, information on
corrosion rate, condition of the electrochemical cell, and mechanisms of the corrosion reactions can be obtained rapidly without requiring a system in a steady state. This method has shown promise in laboratory investigations, but is still an exploratory technique, not sufficiently mature for field application.

4.3.8 Magnetic methods

The main application of magnetic methods is the detection of reinforcing position and size. Pachometers measure cover by detecting variations in an induced magnetic field caused by the ferromagnetic properties of the bars [Bungey 82]. Satisfactory equipment for this purpose is already used in New England. Prototype magnetic instrumentation has also been used for detecting loss of section or fracture of prestressing steel [NCHRPS118 85]. Since essentially all of the region's bridge decks are conventionally reinforced, assessment of prestressing strand condition is not applicable. Magnetic methods were therefore judged not to be of interest for this research project.

4.3.9 Nuclear

The feasibility of using nuclear magnetic resonance (NMR) to determine the moisture content of the concrete and nuclear bombardment to determine chloride content has been investigated, but the methods have not showed promise since they are indirect, expensive, heavy, slow, and NMR requires skilled operators [NCHRPS118 85]. Nuclear bombardment can be used to measure chloride content in the cover concrete by analysis of thermal neutron prompt-gamma- and thermal neutron activation-gamma rays [Westover 84]. The deck is irradiated with a thermal neutron source. The chlorine nuclides emit a characteristic group of prompt-gamma rays upon absorbing a thermal neutron. The decay of the induced chlorine radioactivity emits activation-gamma rays. Measurement of the prompt
and activation gamma rays can be used to find the chloride content profile through the depth of the deck. This method is extremely slow, yielding three to six spot measurements per hour. The capital cost is high and although the FHWA funded development of a prototype instrument, no state highway department has been sufficiently interested to pay fees for demonstration use.

4.3.10 Infrared thermography

Infrared thermography senses the emission of thermal radiation and produces a visual image from this thermal signal. Thermography, like any system using infrared radiation, measures variation in surface radiance and does not directly measure surface temperature [Joyce 84]. It can be used to detect deterioration that is associated with a thermal anomaly. Surface temperature differences develop due to different rates of heat transfer for sound and unsound areas of deck. A delamination in concrete causes a thermal break so that the delaminated area will absorb and emit heat differently than sound concrete. Debonding between the asphalt overlay and the concrete also causes a thermal break. Thermography can identify this temperature difference between the delaminated or debonded area and sound concrete.

Thermography equipment is available to operate under the following conditions: 70 percent sunshine, a dry deck surface, and wind speed below 15 - 20 miles per hour. There is not a restriction on ambient temperature. This van-mounted equipment can survey a full lane width on each pass, moving at a speed of between 2 and 10 miles per hour. A boom elevates the infrared camera approximately 4.3 m (14 ft) above the deck to reduce distortion due to scanning angle and to increase the field of view. The data is analyzed digitally to produce a map of thermal anomalies. An operator must edit the map against a video
recording of the deck to eliminate undesired readings from discoloration, patching, or other surface effects [Kunz 85].

The ability of thermography to detect delaminations on asphalt covered decks is very sensitive to weather conditions such as wind, humidity, and cloud cover [Manning 85]. It is still an open question as to whether or not thermography can reliably discriminate between delaminations and debonding for asphalt-covered decks. Infrared thermography has the advantages of being relatively direct, fast, and high resolution, and the disadvantages of being passive, since it measures re-radiated on solar energy, and sensitive to environmental variables. The potential of this method was deemed to be sufficiently promising to include in this research program.

4.3.11 Ground penetrating radar

Radar techniques, developed from U S Army methods for nonmetallic buried mine detection, have been used to sense defects in concrete since the early 1970s. Radar operates by transmitting high frequency electromagnetic waves and sensing the reflections caused by changes in the electromagnetic properties of the material being probed. Two main types of radar systems have been used in highway surveys: continuous wave, swept frequency modulated radar and short pulse, ground penetrating radar (GPR). GPR is the preferred method for highway applications, due to severe speed restrictions on the swept frequency approach since the transmit frequency variation must be slow enough so that the frequency of the return from the top and bottom of the deck are essentially the same [Joyce 84].

Ground penetrating radar (GPR) is a pulsed microwave method that is capable of detecting anomalies associated with a variety of significant physical conditions. GPR is sensitive to: the location and orientation of the reinforcing steel;
the concrete cover depth to the top mat of reinforcing steel; the asphalt thickness; the moisture content; the chloride content; and the location and extent of deteriorated concrete. It is not sensitive to environmental variables with the exception of moisture on the deck. In a recent test on an asphalt covered deck [Manning 85], radar showed good correlation with known deterioration. There were many false results which showed need for improved signal interpretation. It was judged to offer good potential for a rapid, non-contact, weather independent procedure.

Van-mounted GPR equipment is available which operates at 2 to 10 miles per hour and scans a one foot wide strip for each pass. The FHWA is currently developing a van-mounted radar capable of inspecting decks at 40 miles per hour. Regardless of speed, the extent of coverage desired (the spacing between strips) determines the number of passes required to survey the deck. The data is analyzed digitally to identify deck thickness and reinforcing location [Kunz 85]. Deteriorated areas do produce a distinct signal signature, but additional development of signal processing is required to be able to interpret this data to reliably identify the type, location, and extent of defects.

GPR has the advantages of being nondestructive, non-contact, being an active energy method with controllable input, and being highly developed. The collection of data is rapid so the coverage on a deck can be increased. Radar also has the advantage of being potentially capable of determining deterioration related conditions leading to delamination, such as chloride content, moisture content, and rebar cover [Maser 85b]. Its main disadvantage is that it is an indirect means of determining mechanical properties and hence can be difficult to interpret. The various investigators cited have proposed different interpretive schemes, but no single scheme has been generally accepted. The volume of data produced is
enormous, requiring data reduction and computerized data processing for effective implementation.

4.4 Test methods for further research

Table 4-I summarizes and compares the methods considered. Sonics, optical, rebound, and penetration tests were dropped from further consideration since they are not applicable to assessment of asphalt overlaid decks. Acoustic emission and x/gamma ray methods do not have appropriate commercially available equipment and are not sufficiently mature as field tests. Sonics are not reliable for overlaid decks and ultrasonics require access to both top and bottom of the deck to obtain meaningful signals. Microseismic methods are slow and difficult to interpret. Magnetic methods are slow, yield a narrow amount of information on deck condition, and already are utilized to an appropriate degree. Nuclear method also yield limited data, are slow, and expensive. Radar and thermography have been identified to be the most promising test methods for further research. Infrared thermography has been more fully developed as a commercial technique of demonstrated value for delamination detection on exposed decks. A better definition is needed of the weather and time-of-day windows during which thermography can successfully be applied. Ground penetrating radar has been the favored of the two techniques for research potential because it is less sensitive to ambient conditions, because it is equally effective with or without asphalt overlays, and because of the ease of data acquisition. Improved signal interpretation and automation of data analysis are needed to make radar a routine technique.

The selection of radar and thermography is in concurrence with the conclusions of other surveys of test methods. Recent studies to identify rapid and more generally applicable methods for bridge deck evaluation have been carried out
Table 4-I: Evaluation of test methods

<table>
<thead>
<tr>
<th>TEST METHOD</th>
<th>PAVED DECKS</th>
<th>COMMERC TOP AVAIL.</th>
<th>ACCESS SPEED</th>
<th>OBJECTIVE</th>
<th>WEATHER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hardness</td>
<td>no</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sonics</td>
<td>no</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultrasonecs</td>
<td>yes</td>
<td>yes</td>
<td>no</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Microseismic</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>no</td>
<td>no</td>
</tr>
<tr>
<td>Acoust. emission</td>
<td>yes</td>
<td>no</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>X and gamma ray</td>
<td>yes</td>
<td>no</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Optical</td>
<td>no</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Magnet.</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>no</td>
<td>yes</td>
</tr>
<tr>
<td>Nuclear</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>no</td>
<td>yes</td>
</tr>
<tr>
<td>Thermography</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>Radar</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>no</td>
</tr>
</tbody>
</table>
by the Federal Highway Administration [Joyce 84], the National Cooperative Highway Research Program [NCHRPS118 85], and by the Ontario Ministry of Transport [Manning 83]. These efforts have concluded that among all techniques considered, ground penetrating radar and infrared thermography offer the greatest potential for high speed surveying. These methods are judged to have the most potential for development into routine operational procedures which can be used, in conjunction with existing test practice, to obtain reliable condition assessments of asphalt-covered bridge decks.
Chapter 5

Physical bases and models for thermography and radar

5.1 Infrared thermography

5.1.1 Physical basis

Thermography directly senses emitted infrared radiation. Since there is no calibration to an absolute radiation level, what is detected is the strength and location of thermal anomalies. The behavior of radiant emission is described by the Stefan-Boltzmann law, which states that the power radiated by a body is directly proportional to the fourth power of its absolute temperature [Vanzetti 72]:

$$q_e = \sigma \varepsilon T^4$$

where $q_e$ = total radiant emission of the radiating surface,

- $\sigma$ = Stefan-Boltzmann constant,  
  which is $[5.673 \times 10^{-12} \text{ watts/(cm}^2 \text{ K}^4)]$

- $\varepsilon$ = emissivity factor of the object,  
  which is less than 1.0 which is emissivity of a black body,  
  emissivity of concrete is 0.92 at 20°C [Holt 79],  
  emissivity of asphalt 0.90 to 0.98 [Threlkeld 70], and

- $T$ = absolute temperature of the object (K).

By directly measuring radiant emission ($q_e$), thermography detects differences in surface temperature of the top of the deck ($T_1$). The deck surface temperature is determined by the deck's thermodynamic interaction with its surroundings. Heat flows into and out of the deck by radiation, conduction, and convection [see Figure 5-1].

Radiation is the movement of heat by electromagnetic waves. In addition to
Figure 5-1: Heat transfer mechanisms for a bridge deck

a: Radiation

b: Conduction

c: Convection
emitting radiation to its surroundings, the deck also receives radiation from its surroundings in accordance with the Stefan-Boltzmann law. The ambient temperature \( T_a \) determines the amount of radiation received by the top \( q_{rt} \) and bottom \( q_{rb} \) deck surfaces. The top of the deck also receives radiant energy from the sun. The radiant heat flow is thus made up of emitted and received radiation from the top and bottom of the deck and incident solar radiation at the top of the deck [see Figure 5-1a].

Conduction is the flow of heat through a material from a hot region to a cooler region. Heat flow through a thermally conductive medium is directly proportional to both the temperature difference across the medium and the medium's thermal conductivity in accordance with Fourier's law [Threlkeld 70]. Thermal conductivity is defined as the ratio of the heat flux to the temperature gradient and is measured in heat flow per unit area for a unit temperature difference across a unit thickness [Neville 73]. For one dimensional heat flow through a material layer of uniform thermal conductivity:

\[
q_c = \frac{k}{d} \times (T_+ - T_-)
\]

where \( q_c \) = conductive heat flow through the material

\[
k = \text{thermal conductivity of the material (watts/m/C)}
\]

\[
d = \text{thickness of the layer (m)}
\]

\[
T_+ = \text{temperature of the hotter side (K)}
\]

\[
T_- = \text{temperature of the cooler side (K)}
\]

The conductive heat flow through and idealized deck, neglecting reinforcing and cracking, is thus determined by: the temperature at the top and bottom surfaces, the deck thickness, and the thermal conductivity of the deck itself [see Figure 5-1b].
Convection is the transfer of heat by fluid movement and mixing. Thermal convection takes place between the surrounding fluid air and the solid deck. The convective heat transfer between the deck surface and the surrounding air can be found from Langmuir's equation [Malloy 69]:

\[ q_v = 1.947 \times (T_s - T_a)^{5/4} \times ((v + 0.35) / 0.35)^{1/2} \]

where \( q_v \) = convective heat flow [watts/m²]

\( T_s \) = deck surface temperature [K]

\( T_a \) = ambient air temperature [K], and

\( v \) = air velocity above the near-surface layer [m/sec].

The convection heat flow is thus determined by: the temperature at the top and bottom surfaces, the ambient temperature, and the wind speed [see Figure 5-1c].

Practical heat flow problems usually involve more than one mode of heat transfer. It is not easy to deal with the simultaneous effects of multiple modes since they are nonlinearly related. Conductive heat transfer is proportional to the temperature difference causing it. Convective heat transfer is a function of the temperature difference raised to the 5/4 power. Radiant heat transfer is proportional to the fourth power of the temperature difference. When the driving temperature difference is not too great, a reasonably accurate answer can be obtained for combined mode problems by using a heat transfer coefficient as follows [Ede 67]. The problem is linearized by assuming that the combined heat transfer is proportional to the temperature difference:

\[ q = h \times (T_s - T_a) \]

where \( q \) = combined heat flow
\[
\begin{align*}
\text{h} & = \text{heat transfer coefficient} \\
& \quad \text{[watts/m}^2/\text{C]} \\
T_s & = \text{surface temperature} \\
& \quad \text{[K], and} \\
T_a & = \text{ambient temperature} \\
& \quad \text{[K].}
\end{align*}
\]

The rate of heat flow into, or out of, the deck must be in balance with the sum of the net radiant flow and the convective flow at the deck surface [Halabe 86]. Any imbalance in the heat flows must be absorbed by heating or cooling of the deck material. The change in temperature of a layer of material is directly proportional to the net heat absorbed and inversely proportional to both the volumetric heat of the material and the thickness of the heated layer:

\[
Q = d \times v \times (T_f - T_i)
\]

where \( Q \) = absorbed heat per unit area

\[\text{[joules/m}^2\text{]}\]

\( v \) = volumetric heat of the material

\[\text{[joules/m}^3/\text{C]}\]

\( T_f \) = final temperature of the material

\[\text{[C], and}\]

\( T_i \) = initial temperature of the material

\[\text{[C].}\]

Volumetric heat is the product of a material's specific heat and density and is a measure of the quantity of heat necessary to produce a unit temperature change in a unit volume.

The deck's internal thermal behavior of heat conduction and absorption must balance its radiant and convective interactions with its surroundings. The net radiant flow is a function of: the incident solar radiation, the energy radiated from the air to the deck, and the energy radiated from the deck to the air. The convective flow is a function of: the temperature difference between the air and deck, and the
air velocity. Since the incident solar radiation and ambient air temperature both vary on a daily cycle, the deck's thermal behavior will also fluctuate diurnally.

The physical parameters determining the emitted infrared radiation are thus: deck emissivity, deck surface temperature, ambient air temperature, deck thermal conductivity, deck volumetric heat, thickness of the heated layer, intensity of incident solar radiation, and air velocity.

5.1.2 Deck condition parameters

The fundamental measure obtained with thermography is the strength and location of thermal anomalies. These anomalies may be caused by thermal discontinuities, such as horizontal cracking. In this case, delaminations would have surface temperatures differing from sound concrete. The reported range of surface temperature differences between delaminated and sound deck is 4.5 C (8.1 F) for bare decks under ideal conditions of full summer sun with no wind. The presence of a 3.1 inch thick bituminous overlay reduced this maximum measured difference to 2.0 C (3.6 F) under similar conditions [NCHRPS118 85].

These surface temperature differences arise from different thermal behavior due to different properties of heat conduction and/or heat capacity. The most commonly postulated causal mechanism for the observed temperature difference is an air filled delamination crack acting as a thermal insulator, allowing the delaminated area to heat and cool more rapidly than sound concrete as shown in Figure 5-2 [Kipp 83]. The material which fills a delamination in the field is not necessarily air. Another possible crack-filling substance is water. Water filled cracks would be expected under saturated asphalt overlays in decks without a functional waterproof membrane.

The thermal properties of interest are thus the thermal conductivity and
Figure 5-2: Commonly hypothesized principle of delamination detection

NOTE: AIR FILM BETWEEN DELAMINATION AND SOLID PAVEMENT ACTS AS AN INSULATION.
volumetric heat for concrete, water, and air. The range of these properties is summarized in Table 5-I for materials of interest in deck delaminations. If the delaminations are indeed typically air filled, the substantial difference in thermal conductivity between air and concrete supports expectations of significant temperature differences between a thin delaminated area and the thicker sound deck. Since the thermal conductivity of water is not substantially lower than that of concrete, clear thermal differences at water filled delaminations would not be expected for steady state, conductivity dominated behavior. However, if the deck thermal behavior is dominated by transient effects, the differing thermal capacities of water and concrete would be expected to produce anomalies.

The meaning of the detected magnitude of surface temperature differences has been interpreted in two ways. One postulation [Clemena 78] is that a more severely delaminated spot gives rise to a stronger the thermal anomaly. Severity of delamination is an ill-defined relative measure based on whether cracking in core samples extends completely across the core (severe or medium), or has not yet separated the core into at least two layers (incipient). It is not clear how this vague measure of degree of delamination relates to changed thermal properties of the slab, although it is reasonable to expect the effective thermal conductivity of the delaminated layer to decrease as the number, width, and extent of crack planes increase. The supporting field data does not appear in the referenced paper, but is described as having occasional inconsistencies in the correlation of anomaly strength and severity of delamination.

The second interpretation [Holt 79] is that the magnitude of the surface temperature difference correlates with the depth of the delamination. Larger temperature differences are therefore associated with shallower delaminations. Some field data which supports this interpretation is presented in Table 5-
Table 5-I: Bridge Deck Material Thermal Properties

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>THERMAL CONDUCT.</th>
<th>SPECIFIC HEAT</th>
<th>DENSITY</th>
<th>VOLUMETRIC HEAT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>watt/m°C</td>
<td>J/kg°C</td>
<td>kg/m³</td>
<td>10⁶J/m²°C</td>
</tr>
<tr>
<td></td>
<td>Btu/ft²°F</td>
<td>Btu/ft²°F</td>
<td>lb/ft³</td>
<td>Btu/ft²°F</td>
</tr>
<tr>
<td>Concrete</td>
<td>1.4 to 3.6</td>
<td>840 to 1170</td>
<td>2400</td>
<td>2.016 to 2.808</td>
</tr>
<tr>
<td></td>
<td>0.8 to 2.1</td>
<td>0.20 to 0.28</td>
<td>150</td>
<td>30.0 to 42.0</td>
</tr>
<tr>
<td>[Neville 73]</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Typ. Struct.</td>
<td>2.4</td>
<td>920</td>
<td>2400</td>
<td>2.208</td>
</tr>
<tr>
<td>Concrete</td>
<td>1.388</td>
<td>0.22</td>
<td>150</td>
<td>33.0</td>
</tr>
<tr>
<td>Air</td>
<td>0.0257</td>
<td>1007</td>
<td>1.2</td>
<td>0.0012</td>
</tr>
<tr>
<td></td>
<td>0.015</td>
<td>0.24</td>
<td>0.075</td>
<td>0.018</td>
</tr>
<tr>
<td>[Ede 67]</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>0.60</td>
<td>4180</td>
<td>1000</td>
<td>4.18</td>
</tr>
<tr>
<td></td>
<td>0.35</td>
<td>1.0</td>
<td>62.4</td>
<td>62.4</td>
</tr>
<tr>
<td>[Ede 67]</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>80.3</td>
<td>452</td>
<td>7850</td>
<td>3.55</td>
</tr>
<tr>
<td></td>
<td>46.43</td>
<td>0.108</td>
<td>490</td>
<td>52.9</td>
</tr>
<tr>
<td>[CRC 72AISC]</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asphalt</td>
<td>0.7</td>
<td>840</td>
<td>2100</td>
<td>1.76</td>
</tr>
<tr>
<td></td>
<td>0.4</td>
<td>0.20</td>
<td>131</td>
<td>26.2</td>
</tr>
<tr>
<td>[Hirst 82ASHRAE]</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
II[Manning80]. The deck temperature differentials were thermographically determined at night. After cores were taken to determine delamination thickness, thermocouples were installed 6 mm (0.23") below the deck surface. Daytime thermocouple measurements confirmed that delaminations located near the surface heated more rapidly and to a higher temperature than deeper delaminations. The depths given are shallower than the expected location of the delaminated plane at the top reinforcing layer. If the cover thickness and delamination depth are closely correlated, thicker covers would lead to deeper delaminations and weaker surface temperature differences.

Scaling does not produce a crack plane that could be expected to act as a thermal break. However, the extensive microcracking of the cement matrix may alter the thermal behavior of the concrete sufficiently to establish detectable thermal anomalies. Such behavior has not been substantiated in the literature, although one survey document reports that thermography can be used to identify both delaminations and scaling in asphalt covered deck slabs [NCHRPS118 85].

The thermal properties of concrete vary with the water content of the material, since the conductivity of air and water differ greatly. Table 5-III shows the effect on the thermal conductivity of a normal weight aggregate concrete caused by doubling its volumetric moisture content from 2.5% to 5.0% [Neville 73]. If a membrane failed in one deck area, the moisture content of the concrete would increase. This change in conductivity caused by water infiltration may be lead to thermal anomalies detectable by thermography. The moderate effect of moisture content on thermal conductivity for a particular concrete density, for example only a 10% increase in thermal conductivity for doubling moisture content in Table 5-III, may be too small to generate detectable anomalies.

Thermal conductivity is more sensitive to density change, which might be
Table 5-II: Surface Temperature Differential and Delamination Depth

<table>
<thead>
<tr>
<th>CORE</th>
<th>TEMPERATURE BELOW DECK AVERAGE</th>
<th>AVERAGE DEPTH OF DELAMINATION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C(F)</td>
<td>mm(inch)</td>
</tr>
<tr>
<td>A</td>
<td>0.3(0.5)</td>
<td>solid deck</td>
</tr>
<tr>
<td>B</td>
<td>0.6(1.1)</td>
<td>24(0.9)</td>
</tr>
<tr>
<td>C</td>
<td>1.0(1.8)</td>
<td>21(0.8)</td>
</tr>
<tr>
<td>D</td>
<td>1.8(3.2)</td>
<td>18(0.7)</td>
</tr>
<tr>
<td>E</td>
<td>2.0(3.6)</td>
<td>10(0.4)</td>
</tr>
</tbody>
</table>
related to scaling, than it is to water content. However, density would not be expected to vary much within one deck, so identifiable thermal anomalies would not be expected.

The likelihood of detecting membrane failure or scaling in this way is judged to be low due to the fact that none of the surveyed infrared studies substantiated such behavior. The proposed field testing program should resolve whether or not scaling and membrane failure in asphalt covered decks can be detected by thermography.

Deck conditions such as surface emissivity, asphalt overlay, and concrete cover thickness also influence thermographic readings. Surface texture, spalls, patches, polishing of concrete in wheel paths, debris such as sand, and oil staining all change the emissivity of the deck surface and hence cause thermal anomalies that complicate the identification of delamination [Holt 79].

The presence of an asphalt overlay makes it more difficult to detect deck defects. The overlay is a continuous thermal mass above the delamination and hence damps down the temperature differentials associated with delaminations. The thicker the overlay, the more pronounced this thermal damping becomes. This reduction in the strength of the thermal anomalies tends to mask the desired signal from a delamination and to narrow the tolerable ranges of atmospheric variables.

The presence of an asphalt overlay also introduces the possibility of overlay debonding causing a thermal discontinuity. This complicates signal interpretation since anomalies may not only indicate the significant defect of a delamination but also the incidental presence of debonding. The thermal signature of a delamination may differ from that of a debonded area. A proposed method for distinguishing debonding from delamination identifies delaminations as circular and uniform
Table 5-III: Effect of Moisture Content on Concrete Thermal Conductivity

<table>
<thead>
<tr>
<th>DENSITY</th>
<th>CONDUCTIVITY</th>
<th>CONDUCTIVITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>kg/m³</td>
<td>2.5% M.C.</td>
<td>5.0% M.C.</td>
</tr>
<tr>
<td>lb/ft³</td>
<td>watt/m°C</td>
<td>watt/m°C</td>
</tr>
<tr>
<td></td>
<td>Btu ft/ft² hr°F</td>
<td>Btu ft/ft² hr°F</td>
</tr>
<tr>
<td>1,920</td>
<td>1.056</td>
<td>1.194</td>
</tr>
<tr>
<td>120</td>
<td>0.610</td>
<td>0.690</td>
</tr>
<tr>
<td>2,080</td>
<td>1.315</td>
<td>1.488</td>
</tr>
<tr>
<td>130</td>
<td>0.760</td>
<td>0.860</td>
</tr>
<tr>
<td>2,240</td>
<td>1.696</td>
<td>1.904</td>
</tr>
<tr>
<td>140</td>
<td>0.980</td>
<td>1.100</td>
</tr>
<tr>
<td>2,400</td>
<td>2.267</td>
<td>2.561</td>
</tr>
<tr>
<td>150</td>
<td>1.310</td>
<td>1.480</td>
</tr>
</tbody>
</table>
anomalies and debonds as large, non-circular, and non-uniform. The postulated cause of the non-uniform marbled pattern is the differing amounts of contact between the flexible debonded pavement and the rigid concrete [Kunz 85].

The presence of a waterproofing membrane beneath the overlay is not mentioned in the literature as having any effect on the thermal image. Membranes generally have thermal properties that are similar to asphalt and their minimal thickness makes their contribution to the overall thermal capacity negligible. Membranes in and of themselves would thus be expected to have little thermal effect. However, membranes can allow moisture to puddle under the asphalt and on top of the deck and the felts in older built up membranes could become saturated with water. If water retained on top of the concrete caused anomalous thermal readings, it might complicate the detection of delaminations.

Another deck condition which might affect thermal behavior is the amount of concrete cover. The thickness of the concrete cover over the top reinforcing bars may indirectly affect the strength of the thermal anomalies. Thicker covers are likely to lead to deeper delaminations and hence possibly weaker surface temperature differences.

5.1.3 Environmental parameters

Field tests of thermography have had mixed success. Positive results have generally been verified to validly indicate defects. However, negative results may not indicate lack of defects, but may instead mean that the prevailing conditions were not suitable for defect detection [NCHRPS118 85]. Part of the discrepancy in success rate is due to different definitions of successful delamination detection. One study [Clemena 78] compared the same deck areas for delamination detection by means of: thermography, chain and hammer sounding, and a Delamtest.
was only done in areas where the results of the different methods conflicted. Another study [Kipp 83] selected a test section of a viaduct to confirm 17 delaminated areas detected by thermography. All 17 cores showed delaminations. Another study [Holt 79] used a small amount of coring combined with chain drag to confirm delaminated areas. One investigation [Manning 85] first mapped delaminations with chain drag on the bare deck, then inspected the deck with a variety of methods, including thermography, after a bituminous paving was placed. Thermography identified more than 90 percent of the previously mapped delaminations. Debonding did not cause thermal anomalies. Scaling was not detected, possibly because the scaled areas were all near the curb line. The proposed field testing program has the potential to establish firmer success statistics by comparing thermographic results with systematic coring and, in some cases, complete or partial deck removal.

Infrared thermography is sensitive to a large number of environmental parameters which may confound deck condition measurements either by affecting the quality and clarity of thermal images, or by introducing extraneous anomalies. These confounding parameters can be grouped into two categories, atmospheric variables and those items directly concerned with deck construction and state.

Thermography is affected by the atmospheric conditions of cloud cover, wind speed, and moisture, as well as season of year and time of day. Conditions which give rise to large temperature gradients in the deck produce the largest temperature differentials for sensing deck damage. Large daytime gradients occur for high solar radiation, large ambient temperature and and light wind. Large nighttime reversed gradients occur for sill cloudless nights with large ambient temperature range [Emerson 73]. Cloud cover reduces the amount of solar radiation reaching the deck. Experience has been that a day relatively free of overcast is needed to
adequately define delaminations [Holt 79]. High wind speeds have been reported to affect results in an unpredictable manner, apparently by increased atmospheric attenuation of reflected heat. Recommendations have been made to conduct tests in only mild winds of less than approximately 15 or 20 mph [Manning 80Kipp80]. Tests cannot be made when there is moisture on the deck. The high emissivity of the surface water prevents detection of delaminations [NCHRPS118 85].

More intense and longer solar radiation sets up larger temperature differentials on the deck, making the delaminations stand out more clearly. Various times of day, usually around noon, have been reported as best for testing. One study successfully identified delaminations on a deck with a 3.1 inch asphalt overlay from 11:30 AM to 6:00 PM, with an optimum resolution at 2:00PM [Manning 83]. Summer is the best season for testing since the solar radiation is more intense and the days are longer. Since the deck must be exposed to direct sunlight, stationary shadows or obstructions interfere with tests. Rapidly moving shadows, such as those due to traffic, do not appear to affect results if a filter is used to eliminate the visible light component of radiance reflected from the road surface [Joyce 84].

The combined weather requirements of clear skies, mild wind, dry deck surface, and intense solar radiation limit most testing to the months of April through October in the northern states [Kipp 83]. Average ambient air temperature is not a significant parameter since thermography only measures differences in emitted heat.

5.1.4 Physical Thermal Model of Delamination

A method for obtaining understanding of the interaction of a nondestructive testing technique and a defect in a physical system is to propose a model of the actual system. The model is a simplified description of the defect, excluding features
thought to be irrelevant for the particular test technique. The theory governing the test response can then be applied to the model to generate response signals. If the synthetic signals match field data well, reasonable assurance is provided that the model does indeed incorporate the important features of the actual system. The model is a useful tool for predicting and interpreting signal responses to a variety of conditions. A parametric study using the model provides valuable information on the expected responses and possible limitations of using the technique to detect the defect.

A thermal model of a delaminated bridge deck is shown in Figure 5-3. The deck is split into three layers: a cover layer of sound concrete, a delaminated layer, and a lower layer of sound concrete. The top and bottom layers have thermal properties characteristic of structural concrete. The delaminated layer is a damaged zone with altered thermal properties. The damaged zone is assumed to be approximately 1/4" (0.64 cm) thick. The delamination may be composed of one or more crack planes that are mainly closed under the weight of the overlaying concrete and the traffic loads. The thermal effect of these crack planes is assumed to be equivalent to a uniform thickness horizontal thin open crack, filled with either air or water. The thermal conductivity and volumetric heat of the damaged layer are computed based on this equivalent crack assumption. The deck properties which must be specified are: the thickness of the cover, delamination, and lower deck layers; the thermal conductivity and volumetric heat of the sound and damaged concrete; and the heat transfer coefficients for the top and bottom of the deck.

The environmental conditions of ambient temperature and incident solar radiation drive the thermal response of the model. Both of these quantities vary on a diurnal cycle as illustrated in Figure 5-4 [Threlkeld 70]. Since the driving forces are a function of time, the system is not static. The system could be treated as if it
Figure 5-3: Physical thermal model of delamination
were static if the deck response time to variations in the driving forces were short compared to the length of time over which the driving forces fluctuated. This condition for quasi-static behavior is not satisfied for the deck model since the thickness and volumetric heat of the concrete give the deck significant thermal inertia, slowing the system's response time. The thermal model must therefore account for transient heat flow behavior. The non-quasi-static character of the model is verified in one of the example cases presented below.

The behavior of the postulated thermal delamination model was explored using a finite element program with capabilities for transient heat flow analysis [Ghaddar 86NEKTON2]. The following eight bare deck example cases were analyzed:

- **Case 1**: Thick(15", 38.1cm) sound deck with no delamination damage: compare with reported actual field behavior.

- **Case 2**: Thin(7.5", 19.1cm) sound deck with no delamination damage: establish base case for comparison with defect cases.

- **Case 3**: Delaminated layer with thermal properties equivalent to a 0.010" (0.25 mm) air-filled crack, thin deck with 1.5" (3.8cm) cover: explore most commonly postulated mechanism.

- **Case 4**: Delaminated layer with thermal properties equivalent to a 0.03" (0.76mm) air-filled crack, thin deck with 1.5" (3.8cm) cover: determine effect of increased equivalent crack width.

- **Case 5**: Delaminated layer with thermal properties equivalent to a 0.05" (1.27mm) air-filled crack, thin deck with 1.5" (3.8cm) cover: determine effect of further increased equivalent crack width.

- **Case 6**: Delaminated layer with thermal properties equivalent to a 0.05" (1.27 mm) water-filled crack, thin deck with 1.5" (3.8cm) cover: determine effect of a different crack-filling substance.

- **Case 7**: Delaminated layer with thermal properties equivalent to a 0.05" (1.27mm) air-filled crack, thin deck with 2.0" (5.1cm) cover: investigate effect of increased cover.
Figure 5-4: Diurnal ambient temperature and solar intensity

Typical variations of outdoor air temperature and intensity of solar radiation incident upon a horizontal surface for a location at 42 deg north latitude on July 1 for a clearness number of unity.
• Case 8: Delaminated layer with thermal properties equivalent to a 0.05" (1.27mm) air-filled crack, thin deck with 2.5" (6.4cm) cover: investigate effect of further increased cover.

The thermal properties assumed for the eight examples are listed in Table 5-IV. Effective thermal properties for the damaged layers were computed using the following formulas:

\[
\frac{1}{k_{\text{effective}}} = \frac{1}{d_{\text{effective}}} \sum \left( \frac{d_i}{k_i} \right)
\]

\[
\frac{1}{\nu_{\text{effective}}} = \frac{1}{d_{\text{effective}}} \sum \left( \frac{d_i}{\nu_i} \right)
\]

The simplified functions which were used for the driving forces are shown in Figure 5-5a and 5-5b. The ambient temperature was assumed to vary sinusoidally and the incident solar radiation was assumed to vary parabolically [Collingbourne 75]. The finite element grids and input files for the eight example problems are presented in Appendix A.

Figure 5-6 [Manning 83] shows field data for temperature variation over a 24 hour period in the top 260 mm (10 in.) of a thick slab deck with an exposed concrete surface in Ontario on a summer day under clear skies. Figure 5-7 shows the results of the Case 1 model of a sound deck.

The Case 1 data is in general agreement with the field data, although the nocturnal thermal profile through the deck is much more uniform for the analytic model results. This difference is believed to be due to a greater slab thickness in the field data. Exact agreement between the field and model data is not to be expected since detailed information was not available on the conditions of the field test and a large number of assumptions were made in setting the model parameters for material thermal properties, deck configuration, and driving forces. The shape and character of the field and model curves are similar, so the model does indeed appear to capture the basic physical behavior.
Table 5-IV: Thermal properties for modeled examples

<table>
<thead>
<tr>
<th>CASE</th>
<th>$d_p$</th>
<th>$d_D$</th>
<th>$d_B$</th>
<th>$k_e$</th>
<th>$v_e$</th>
<th>$k_B$</th>
<th>$v_B$</th>
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<tr>
<td></td>
<td>cm</td>
<td>cm</td>
<td>cm</td>
<td>watt/m°C</td>
<td>$10^6$ J/m$^3$C</td>
<td>watt/m°C</td>
<td>$10^6$ J/m$^3$C</td>
</tr>
<tr>
<td></td>
<td>inch</td>
<td>inch</td>
<td>inch</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>1</td>
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<td>33.655</td>
<td>2.40</td>
<td>2.208</td>
<td>2.40</td>
<td>2.208</td>
</tr>
<tr>
<td></td>
<td>1.50</td>
<td>0.250</td>
<td>13.250</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
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<td>0.635</td>
<td>14.605</td>
<td>2.40</td>
<td>2.208</td>
<td>2.40</td>
<td>2.208</td>
</tr>
<tr>
<td></td>
<td>1.50</td>
<td>0.250</td>
<td>5.750</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>3.81</td>
<td>0.635</td>
<td>14.605</td>
<td>2.40</td>
<td>2.208</td>
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<td>0.0296</td>
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<td>5.750</td>
<td></td>
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<td>14.605</td>
<td>2.40</td>
<td>2.208</td>
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<td>14.605</td>
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<td>2.208</td>
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<td>0.0060</td>
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<tr>
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<td>1.50</td>
<td>0.250</td>
<td>5.750</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>3.81</td>
<td>0.635</td>
<td>14.605</td>
<td>2.40</td>
<td>2.208</td>
<td>1.500</td>
<td>2.438</td>
</tr>
<tr>
<td></td>
<td>1.50</td>
<td>0.250</td>
<td>5.750</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>5.08</td>
<td>0.635</td>
<td>13.335</td>
<td>2.40</td>
<td>2.208</td>
<td>0.123</td>
<td>0.0060</td>
</tr>
<tr>
<td></td>
<td>2.00</td>
<td>0.250</td>
<td>5.250</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>6.35</td>
<td>0.635</td>
<td>12.065</td>
<td>2.40</td>
<td>2.208</td>
<td>0.123</td>
<td>0.0060</td>
</tr>
<tr>
<td></td>
<td>2.50</td>
<td>0.250</td>
<td>4.750</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 5-5: Simplified thermal driving forces

Figure 5-5a sinusoidal ambient temperature

Figure 5-5b parabolic incident solar radiation
Figure 5-6: Thermal field data for sound thick slab
Figure 5-7: Thermal Case 1: sound thick slab
Having established the general validity of the model, a 7.5" sound deck was analyzed in Case 2 to set a base line for an undamaged slab of typical thickness. Figure 5-8 shows that the day time behavior of the thin slab is similar in character to that of the thick slab. The thin slab did not develop the expected inverted nighttime profile, but instead developed almost uniform temperatures through the deck. apparently due to the lower heat capacity of the thinner deck. As can be seen in Figure 5-9, the top of slab temperature for the thicker slab was moderately higher during the day time, since the heat capacity of the thicker slab kept it from becoming as cool as the thin slab during the early morning predawn hours.

To verify that the deck behavior cannot be reproduced by static thermal analysis, comparable temperature profiles for a steady state hand-solution and the Case 2 transient solution are shown in Figure 5-10. The steady-state solution yields unrealistically high temperatures since the length of time necessary for the deck to reach these temperatures under the specified incident radiation level is not taken into account in a steady-state approach. Figure 5-11 shows how the temperature profile through a thin deck varies through the day. The strong noon gradient becomes moderate by 4:00PM and reverses by 8:00PM. Both top and bottom of the deck cool more than the core during the night. At sunrise the gradient is reestablished, moderately at 8:00AM and increasing in strength as noon approaches.

Cases 3, 4, and 5 model a delamination as a damaged zone having thermal properties equivalent to the presence of a thin air-filled crack. Figure 5-12 [Manning 80] shows field data for near surface temperatures of solid and delaminated concrete at midday in the summer for a thin slab deck. Figure 5-13 shows comparable curves for the solid thin model Case 1 and the 0.05" air-filled crack model Case 5. The delaminated deck is hotter than the solid deck by 3C
Figure 5-8: Thermal Case 2: sound thin slab
Figure 5-9: Thermal Cases 1 and 2: compare thick and thin slabs
Figure 5-10: Verify dominance of transient thermal behavior
Figure 5-11: Thermal cycle of a thin deck
maximum for the field data and 4C for the analytic data. The analytic model shows a significant temperature difference earlier in the day and does not begin cooling immediately after noon, but is in general agreement with the behavior shown by the field data. Figure 5-14 shows the complete daily cycle for Cases 2 and 5. The delaminated deck modeled in Case 5 cooled more than the solid slab at night, but the nocturnal temperature differences between sound and delaminated deck were quite small.

The width of the air-filled crack was found to influence the size of the temperature difference as is shown in Figure 5-15. Increasing the crack width by a factor of five increased the maximum surface temperature difference by a factor of four. The crack width did not affect the times at which the delaminated deck temperature crossed the solid deck temperature in the morning near sunrise and in the evening near sunset.

Case 6 explored the effect of crack filler material on deck surface temperature. Figure 5-16 shows that a 0.05" water-filled crack does not produce a significant difference from a solid deck in deck surface temperature. Figure 5-17 shows the very small difference in surface temperature between the Case 2 solid deck and the Case 6 water-filled crack, in sharp contrast to the strong temperature differences produced by the air-filled cracks of Cases 3 and 5.

Cases 7 and 8 investigated the effect of varying cover thickness on the thermal anomaly produced by an air-filled delamination. Increasing cover reduced the size of the temperature difference between solid and delaminated areas as shown in Figure 5-18. Increasing cover thickness also had a very weak effect on when the delaminated temperature crossed the solid temperature.

Overall, the results of these six cases show that the proposed physical
Figure 5-12: Temperature difference, solid versus delaminated: field data
Figure 5-13: Temperature difference, solid versus delaminated: Cases 2, 5
Figure 5-14: Thermal Case 2, 5: solid deck compared to 0.05" air-filled crack
Figure 5-15: Thermal Cases 2, 3, 4, 5: effect of crack width
Figure 5-16: Thermal Cases 2, 6: solid deck compared to 0.05" water-filled crack
Figure 5-17: Thermal Cases 2, 3, 5, 6: effect of crack filler material
Figure 5-18: Thermal Cases 2, 5, 7, 8: effect of cover thickness
thermal model has promise for explaining the thermal response of a delaminated slab.

5.2 Ground penetrating radar

5.2.1 Physical basis

Ground penetrating radar operates by directing a pulsed radar signal into the tested material and detecting the arrival time and magnitude of reflected electromagnetic waves. The source and receiver radar antennae are both located above the deck. The waves are reflected at a boundary between two materials with different dielectric constants. Returns would be expected from the following interfaces in a typical bridge deck: air/asphalt, asphalt/concrete, concrete/top steel, concrete/bottom steel, and concrete/air. Figure 5-19 [Maser 85a] illustrates the propagation of the transmitted pulse into the deck and the reflections generated at the dielectric interfaces.

The amount of electromagnetic energy that is reflected at an interface between two media is determined by their wave impedances:

\[ r_{12} = \frac{Z_2 - Z_1}{Z_2 + Z_1} \quad [\text{Steinway 81}] \]
\[ t_{12} = \frac{2Z_2}{Z_2 + Z_1} \quad [\text{Ramo 84}] \]

where \( r_{12} \) = reflection coefficient between medium 1 and medium 2
\( t_{12} \) = transmission coefficient between medium 1 and medium 2
\( Z_1 \) = wave impedance of medium 1 (ohm), and
\( Z_2 \) = wave impedance of medium 2 (ohm).

The reflection coefficient is the fraction of the incident wave amplitude which is reflected back into medium 1 at the interface. The transmission coefficient is the fraction of the incident wave amplitude which travels through the boundary into medium 2 (see Figure 5-20).
Figure 5-19: Radar reflections in an overlaid bridge deck
Figure 5-20: Reflection of radar wave at dielectric boundary
The wave impedance is zero for a perfect conductor. The impedance for a non-metallic material is [Clemena 85]:

\[ Z = \frac{Z_0}{\sqrt{\varepsilon_r}} \]

where \( Z_0 \) = wave impedance for free space (air), which is 376.1 ohm, and \( \varepsilon_r \) = relative dielectric constant.

A material's dielectric constant, also known as dielectric permittivity, is the amount of electrostatic energy stored per unit volume for a unit potential gradient. The ratio of a material's dielectric constant to that of free space is defined as the relative dielectric constant [Clemena 85]:

\[ \varepsilon_r = \frac{\varepsilon}{\varepsilon_0} \]

where \( \varepsilon \) = dielectric constant (farad/meter), and \( \varepsilon_0 \) = dielectric constant of free space (air), which is \( 8.85 \times 10^{-12} \) farad/meter.

The concrete and bituminous paving used in bridge decks are not single phase media. They may be viewed as multiphase materials consisting of a solid matrix with air and liquid together filling the pore space. Assuming the three substances act independently, the following expression may be used to calculate the average dielectric constant for the solid/air/liquid mixture [XADAR]:

\[ \varepsilon_{rm}^{1/2} = \{(1 - \pi) \ast \varepsilon_{rs}^{1/2}\} + \{S \ast \pi \ast \varepsilon_{rl}^{1/2}\} + \{(1 - S) \ast \pi \ast \varepsilon_{ra}^{1/2}\} \]

where \( \varepsilon_{rm} \) = relative dielectric constant of the mixture, \( \varepsilon_{rs} \) = relative dielectric constant of the solid, \( \varepsilon_{rl} \) = relative dielectric constant of the liquid, \( \varepsilon_{ra} \) = relative dielectric constant of air, which is 1.0, \( \pi \) = porosity of the solid, and \( S \) = fractional liquid saturation.

The reflection coefficient can be expressed in terms of relative dielectric constants instead of wave impedances:
\[
\begin{align*}
  r_{12} &= \left( \frac{\epsilon_{r1}^{1/2} - \epsilon_{r2}^{1/2}}{\epsilon_{r1}^{1/2} + \epsilon_{r2}^{1/2}} \right) \\
\end{align*}
\]
where \( \epsilon_{r1} \) = relative dielectric constant of medium 1, and \( \epsilon_{r2} \) = relative dielectric constant of medium 2.

Values for electromagnetic properties used in bridge decks are given in Table 5-V [Ulriksen 82]. Using the tabulated relative dielectric constants, Table 5-VI shows how reflection coefficients vary with the relative dielectric constant of concrete for interfaces in a bridge deck. Since steel is a good conductor with a wave impedance of zero, its relative dielectric constant is infinite and the steel interface acts as a perfect reflector. When the relative dielectric constant of medium 1 is smaller than that of medium 2, the reflection coefficient is negative, indicating a phase reversal of the reflected wave. This can be seen in Figure 5-20, and in Figure 5-19 where all the radar reflections have a reversed phase from the transmit pulse except the bottom deck return. This agrees with the reflection coefficients in Table 5-VI, which are all negative except the concrete/air coefficients at the bottom of the deck.

In addition to describing the reflection of a radar wave at a dielectric boundary, the propagation of a wave as it passes through a uniform medium must be described in order to understand the behavior of a radar wave traveling through a bridge deck.

The velocity at which a wave propagates in a dielectric medium may be calculated from the expression [Clemena 85]:

\[
v = \frac{c}{\epsilon_r^{1/2}}
\]

where \( v \) = velocity in medium \( \text{(m/sec)} \)

\( c = \text{velocity in free space} \ (3 \times 10^8 \ \text{m/sec}), \) and

\( \epsilon_r = \text{relative dielectric constant}. \)

Propagation velocities for various deck materials are shown in Table 5-V.
**Table 5-V:** Electric parameters of bridge construction materials

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>ε_r</th>
<th>Z</th>
<th>σ</th>
<th>ν</th>
<th>λ_m</th>
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<td></td>
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<td>mho/m</td>
<td>m/nsec</td>
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<tr>
<td>Air</td>
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<td>377</td>
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<td>0.30</td>
<td>0</td>
</tr>
<tr>
<td>Water, pure, 18°C</td>
<td>81</td>
<td>42</td>
<td>3.7*10⁻⁶</td>
<td>0.03</td>
<td>7.75*10⁻⁵</td>
</tr>
<tr>
<td>Ice, freshwater</td>
<td>4</td>
<td>188</td>
<td>1.0*10⁻³</td>
<td>0.15</td>
<td>9.43*10⁻²</td>
</tr>
<tr>
<td>Concrete, strong, dry</td>
<td>6.0</td>
<td>154</td>
<td>1.2*10⁻³</td>
<td>0.12</td>
<td>9.23*10⁻²</td>
</tr>
<tr>
<td>Concrete, strong, soaked 20 hrs in freshwater</td>
<td>10.9</td>
<td>114</td>
<td>7.1*10⁻³</td>
<td>0.09</td>
<td>4.05*10⁻¹</td>
</tr>
<tr>
<td>Concrete, cracked, dry</td>
<td>4.5</td>
<td>177</td>
<td>2.25*10⁻³</td>
<td>0.14</td>
<td>2.0*10⁻¹</td>
</tr>
<tr>
<td>Concrete, cracked, soaked 20 hrs in freshwater</td>
<td>13.7</td>
<td>102</td>
<td>3.2*10⁻³</td>
<td>0.08</td>
<td>1.63*10⁻¹</td>
</tr>
<tr>
<td>Asphalt, cracked, soaked 20 hrs in freshwater</td>
<td>2.0</td>
<td>267</td>
<td>1.25*10⁻³</td>
<td>0.21</td>
<td>1.67*10⁻¹</td>
</tr>
</tbody>
</table>
Table 5-VI: Variation of reflection coefficients with concrete dielectric constant

<table>
<thead>
<tr>
<th>INTERFACE</th>
<th>APPROXIMATE REFLECTION COEFFICIENT FOR CONCRETE RELATIVE DIELECTRIC CONSTANT</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon_r$ concre</td>
<td>4</td>
</tr>
<tr>
<td>Air/asphalt</td>
<td>-0.17</td>
</tr>
<tr>
<td>Asphalt/concrete</td>
<td>-0.27</td>
</tr>
<tr>
<td>Concrete/steel</td>
<td>-1.00</td>
</tr>
<tr>
<td>Concrete/water</td>
<td>-0.64</td>
</tr>
<tr>
<td>Concrete/air</td>
<td>0.33</td>
</tr>
</tbody>
</table>
The attenuation of the wave energy as it propagates through a dielectric medium may be obtained from the following approximate expression for the materials of interest [Maser 85b]:

\[ A_m = \left[ \frac{\sigma_m^2}{4 \epsilon_{rm}} \right]^{1/2} \times 377 \text{ for } \sigma_m / \epsilon_{rm} \ll 1 \]

where \( A_m \) = attenuation in medium m \( (m^{-1}) \),
\( \sigma_m \) = conductivity in medium m \( (\text{mho/m}) \),
\( \epsilon_{rm} \) = relative dielectric constant of medium \( (m) \), and
\( \omega \) = wave frequency \( (\text{Hz}) \).

In this range, attenuation is independent of frequency. Values for attenuation are given in Table 5-V. Attenuation is a strong function of conductivity and a weaker function of permittivity. As can be seen in Table 5-V, the conductivity of deck materials vary much more widely than their permittivity. For these reasons, attenuation behavior is dominated by conductivity.

Electrical conductivity is a measure of the ease with which an electrical current can be made to flow through a material. Electrical resistivity is the reciprocal of electrical conductivity. As noted previously, concrete and paving are not single materials but are rather multiphase mixtures. The conductivity of a saturated mixture of insulating particles in an electrolytic medium may be found from the empirical relationship called Archie's law [McNeill 80]:

\[ \frac{\sigma_x}{\sigma_1} = n^m \]

where \( \sigma_x \) = conductivity of mixture \( (\text{mho/m}) \),
\( \sigma_1 \) = conductivity of electrolyte \( (\text{mho/m}) \),
\( n \) = fractional porosity of the mixture, defined as the ratio of the volume of the electrolytic solution to the total volume of the mixture, and
\( m \) = empirical parameter.

Archie's law only applies to fully saturated mixture, a condition not usually satisfied by bridge deck materials. However, the following similar relationship approximately expresses the conductivity of unsaturated mixtures [McNeill 80]:
\[ \sigma_x / \sigma_1 = s^p \]

where \( s \) = fraction of solution, the ratio of the volume filled with electrolyte to the total volume of the mixture, and \( p \) = parameter experimentally determined to be approximately 2.

Ground penetrating radar thus directly detects the arrival time and magnitude of a reflected pulsed electromagnetic signal. Since the time and strength of the transmitted signal are known, the time of flight and attenuation of each reflection may be found. With this information the pulse velocity, location of boundaries, and reflection coefficients at boundaries can be computed. The received signal thus contains information on what was reflected, how quickly the signal traveled, and how much the signal was attenuated. These quantities are dependent on the electrical properties of the deck materials.

5.2.1.1 Deck condition parameters

Radar detects the arrival time and energy level of a reflected electromagnetic pulse. Since radar is affected by changes in dielectric properties, variations in deck condition and design will cause changes in the signal. Information is obtained by observing the return time, amplitude, shape, and polarity. An idealized waveform is shown in Figure 5-21 [Maser 85b]. The interface causing each return of the transmit pulse are labeled. The first full cycle return made up of the first negative and first positive peaks, is actually an overlapping of the two returns from the air/asphalt and asphalt/concrete interfaces, as is discussed in more detail in Section 5.2.2. The shape of the waveform depends on the arrival time and amplitude of the returns. If we assume the deck is made of layers with constant dielectric properties the arrival times and amplitudes may be simply calculated.

If the thickness and velocity for each layer is known, calculating the return time is a straight forward summation of the time needed to penetrate and return
Figure 5.21: Idealized radar waveform
through each layer. For example, the return time for the asphalt/concrete interface is:

\[ T_{total} = t_{air} + t_{asphalt} = 2 \times \left[ \frac{x_{air}}{v_{air}} + \frac{x_{asphalt}}{v_{asphalt}} \right] \]

where
- \( T_{total} \) = time between transmission and return
- \( t_{air} \) = round trip travel time in air
- \( t_{asphalt} \) = round trip travel time in asphalt
- \( x_{air} \) = height of antenna above pavement
- \( x_{asphalt} \) = thickness of asphalt
- \( v_{air} \) = wave velocity in air
- \( v_{asphalt} \) = wave velocity in asphalt.

If the relative permittivities (and thus the velocities) of the materials are known the thicknesses of the various layers can be calculated from the measured flight times of the reflected signals. Conversely, the permittivities can be found if the thicknesses are known. Radar can thus be used to determine the thicknesses of the asphalt overlay and the concrete cover.

Calculation of return amplitudes is based on the cumulative action of the reflection coefficients at the interfaces and the attenuation within each layer. Figure 5-22 show the calculation of return amplitudes for an idealized uncracked deck. Figure 5-23 shows the amplitudes for cracked deck with attenuation neglected.

The signal will be reflected at each interface within the deck. However, returns from the top and bottom of a thin horizontal crack are not detectable since their reflections are of opposite polarity (top and bottom reflection coefficients are of opposite sign) and overlap (return times are essentially identical), hence canceling each other. The thickness of the delamination voids to be detected is as small as 0.04" (1.0mm) [Joyce 84]. Air-filled cracks in concrete less than 0.1 inches wide
Figure 5.22: A return amplitude for an uncracked deck, accounting for attenuation.

\[ \rho_{i,a} \rho_{j,b} \rho_{k,c} \]

\[ \rho_{i,a}^{\prime} \rho_{j,b}^{\prime} \rho_{k,c}^{\prime} \]

Layer 1: Asphalt
Layer 2: Cover
Layer 3: Concrete

Transmission coefficients:
- \( t_x \) = transmission coefficient travelling down through interface \( x \)
- \( t_x^* \) = transmission coefficient travelling up through interface \( x \)

\( \rho = e^{-\mu (\text{layer thickness})} \)

\( \mu = \left\{ \begin{array}{ll} \frac{\alpha}{\delta} & \text{fluid} \\ \frac{\alpha}{\delta c} & \text{solid} \end{array} \right\} \)

Top bars: \( 2\rho_{i,a}^{\prime} \rho_{j,b}^{\prime} \cdot a_l \)
Bottom bars: \( 2\rho_{i,a}^{\prime} \rho_{j,b}^{\prime} \rho_{k,c}^{\prime} \cdot a_j \)

Note: Rebar returns are approximate due to neglecting oblique paths, and exact depth of bottom bars.
Figure 5-23: Return amplitudes for a cracked deck, neglecting attenuation
would not be expected to produce a directly detectable reflection. Even though the crack reflections cancel, they still reduce the energy of the wave which propagates past the crack. Presence of a thin air-filled crack may therefore be indirectly detectable based on the attenuation of energy reflected from interfaces beneath the crack. If the crack is filled with water instead of air, the signal will take about nine times longer to travel across the crack. This will separate the time between the returns from the top and bottom of the crack and reduce the signal cancellation. Water-filled cracks may therefore be directly observable and will attenuate the returns from below the crack even more than if the crack was air-filled [Maser 85b].

Returns would not be expected from the membrane beneath the asphalt since the tars, rubbers, and plastics used for membranes [Frascoia 77] tend to have dielectric properties that are similar to the asphalt, as can be seen from Table 5-VII [CRC 72]. Any weak returns would also tend to be self-canceling like the returns from a thin crack.

The attenuation of signal strength is a strong function of conductivity. The moisture content has a strong effect on the conductivity of concrete or asphalt. Using Archie's law modified for unsaturated mixtures [McNeill 80], the effect of moisture content on concrete conductivity is as shown in Figure 5-24. A value of 20,000 ohm-cm [Hope 85] was chosen as representative for uncontaminated concrete resistivity (conductivity = 5 x 10^{-5} mho/cm = 5 x 10^{-3} mho/m) with an assumed moisture content of 5% (see Appendix B). Attenuation is a weaker function of relative permittivity, which is a direct function of the fractional liquid saturation of a mixture [XADAR]. The amount of signal attenuation can thus provide information on the deck moisture. If moisture content of the deck varied widely depending on membrane condition, it might be feasible to detect membrane leakage using radar.
Table 5-VII: Dielectric Constants

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>TEMPERATURE</th>
<th>FREQUENCY</th>
<th>DIELECTRIC CONSTANT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polyethylene</td>
<td>23</td>
<td>$1 \times 10^3 - 1 \times 10^8$</td>
<td>2.26</td>
</tr>
<tr>
<td>Butyl Rubber</td>
<td>25</td>
<td>$1 \times 10^3 - 1 \times 10^8$</td>
<td>2.42 - 2.39</td>
</tr>
<tr>
<td>Asphalt</td>
<td>17 - 22</td>
<td>$&lt;3 \times 10^6$</td>
<td>2.68</td>
</tr>
<tr>
<td>Polyvinyl Chloride</td>
<td>25</td>
<td>$1 \times 10^6$</td>
<td>3.30</td>
</tr>
<tr>
<td>Epoxy Resins</td>
<td>25</td>
<td>$1 \times 10^6$</td>
<td>3.52 - 3.62</td>
</tr>
<tr>
<td></td>
<td></td>
<td>hertz</td>
<td></td>
</tr>
</tbody>
</table>
Figure 5.24: Effect of moisture content on concrete conductivity
The conductivity of concrete varies proportionally with the amount of salt in solution within the pore space [Ulriksen 82], since the amount of salt in the pore water strongly influences the conductivity of the liquid. The following equation describes the effect of salt ions on the conductivity of a dilute solution at normal temperatures [McNeill 80], as shown in Figure 5-25:

\[
\sigma_s = 96500 \times \sum C_i M_i
\]

where \( \sigma_s \) = conductivity of solution (mhos/m)
\( C_i \) = number of gram equivalent weights of \( i^{th} \) ion per \( 10^6 \) cm³ of water
\( M_i \) = mobility of \( i^{th} \) ion (m/sec/volt/m)

\( = 1/58 \) for both sodium and chlorine, and

\( = 5.2 \times 10^{-8} \) for sodium
\( = 7.9 \times 10^{-8} \) for chlorine.

The strength of the effect of chloride content on concrete conductivity depends on the amount of moisture in the mixture as shown in Figure 5-26 (see Appendix B). Based on the proportional relationship of concrete conductivity to chloride content, the attenuation of radar returns can be used to determine the chloride content of decks. Since the moisture content of concrete is proportional to the relative permittivity, increasing the moisture content of the cover concrete increases the reflection coefficient of the asphalt/concrete boundary. Utilizing these facts, a recent study [Ulriksen 82] characterized scaling as having high chlorides and moisture and used the ensuing high attenuation and enhanced reflection coefficient to detect scaling damage.

5.2.1.2 Environmental parameters

Atmospheric conditions have little effect on radar tests, so this method is largely independent of weather. Rising temperature affects the conductivity of asphalt and concrete by increasing the conductivity of the liquid portion of the
Figure 5-25: Effect of dissolved salt on water conductivity
Figure 5-28: Effect of salt content on concrete conductivity
mixture. The variation of the conductivity of the solution is approximately linear over the normal ambient temperature range and is given by the following expression [McNeill 80]:

\[ \sigma_T = \sigma_{25C} \times [1 + \beta(T - 25)] \]

where \( \sigma_T \) = conductivity of solution at \( T \) (mhos/m),
\( T \) = temperature at which conductivity is to be calculated (C), and
\( \sigma_{25C} \) = conductivity of solution at 25C
\( \beta \) = temperature coefficient for ion solution (C\(^{-1}\))
\( \beta = 0.022 \) per C for sodium chloride solution.

Figure 5-27 shows the effect of temperature on concrete conductivity. A temperature change from 7C to 34C, a rise of almost 30C, is required to double the concrete conductivity.

Water strongly attenuates radar signals so testing cannot be done when the deck surface is actually wet. Since attenuation of the radar signal increases with both increasing moisture content and temperature, it has been suggested that radar would not be effective for subpavement road inspection applications if there is both high moisture and high temperature, due to insufficient energy to overcome the high attenuation [Steinway 81]. It has also been postulated that significant moisture in the overlay would cause a similar difficulty for deck inspection applications [NCHRPS118 85]. Actual attenuation for expected ranges of moisture and temperature for deck inspection applications is an area requiring further investigation and is further addressed in Section 5.2.2.

5.2.2 Physical Radar Model of Deck Defects

A physical model of the deck's radar behavior can be used to explore the expected waveform responses to various deck conditions. An existing computer model [Maser 85b] was used to test some of the hypotheses concerning the
Figure 5-27: Effect of temperature on concrete conductivity
waveforms generated by various deck conditions and environmental parameters. The analytic model uses a multilayered idealization of the bridge deck where each layer has constant dielectric properties. The physical model is composed of an asphalt layer, a top concrete layer, a crack layer 0.05" wide, a bottom concrete layer, and top and bottom reinforcing mats. The relative dielectric permittivity and the electric conductivity of the asphalt, cover concrete, crack, and bottom concrete layers may all be independently set. The reinforcing bars are modeled as perfectly reflective cylinders, with the specified location, diameter, and spacing.

The computer program SYNTH\(^1\) [Maser 85b] uses the specified layer and rebar parameters to synthesize a radar waveform. An example synthetic waveform is shown in Figure 5-21. Figure 5-28 [Chung 84] shows an actual waveform from a sound concrete deck with an asphalt overlay. The synthesized waveform has a similar overall shape as the actual waveform, indicating that the model captures at least an approximation of the actual deck radar response behavior.

The amplitudes and arrival times of selected peaks, the timing of zero crossings, and the absolute value of the integral between selected times have been used to summarize the characteristics of a waveform in a concise manner suitable for making the comparisons and generalizations necessary for signal interpretation.

---

\(^1\)The code for this computer program contains a bug. The transmission coefficients \(t_{12}\) and \(t_{21}\) are both computed as \(1 - |r_{12}|\). The correct values of the transmission coefficients are \(t_{12} = 2Z_2/(Z_2 + Z_1)\) and \(t_{21} = 2Z_1/(Z_1 + Z_2)\). The effect of this error is to excessively reduce the amplitude of the detected signal for each interface through which the pulse travelled. Since each additional layer compounds the error, the portion of the waveform containing signals from the lower layers of the deck have amplitudes which are much too low. All waveforms which were generated using SYNTH have this error so all amplitudes below the first interface are distorted.
Figure 5-28: Radar waveform from sound concrete deck
The chosen summarizing set is called the "signature" of the waveform. The signature used here is the one used in the study for which SYNTH was written [Maser 85b]. The selected waveform characteristics are shown in Figure 5-21. The signature peaks are:

1 = first positive peak corresponding to the top of concrete return
2 = second negative peak corresponding to the top rebar return
3 = third negative peak corresponding to the top rebar return
4 = fourth negative peak corresponding to the bottom of deck return

The first positive peak, representing the second half cycle of the asphalt/concrete interface return, was chosen since the first response cycle is made up of overlapping returns from the air/asphalt and asphalt/concrete boundaries. The first negative peak tends to be characteristic of the air/asphalt interface while the first positive peak tends to be representative of the asphalt/concrete boundary. The arrival time and amplitude of peak i are referred to as $t_i$ and $a_i$, respectively, except for $a_1$ which refers to the amplitude of the first negative peak. To construct a signature, $a_2$, $a_3$, and $a_4$ are normalized by $a_1$ to use a consistent measure of the energy arriving at the pavement surface. The time parameters of the signature are adjusted. Differences in the peak arrival times and are indicative of the time of flight of the signal through the various deck layers. Figure 5-21 shows the resulting signature composed of the six numbers: A1, A2, A3, T1, T2, T3.

Waveforms were synthesized for 27 cases which can be grouped into five classes:

- Group I, made up of nine cases, examined the effect of varying cover thickness on solid deck, deck with a water-filled delamination crack, and deck with an air-filled delamination crack.

- Group II, made up of five cases, modeled progressive membrane failure by varying the moisture of the three layers asphalt/cover/concrete from wet/dry/dry to wet/wet/wet.
• Group III, made up of four cases, explored the effect of increasing chloride contamination in the asphalt and cover layers.

• Group IV, made up of six cases, looked at the combined effects of moisture and temperature.

• Group V, made up of three cases, looked at the effect of varying asphalt thickness.

The details of the individual synthesized cases are presented in Appendix B.

The objective of the first group of cases was to find what signature parameters were indicative of delamination damage. Delamination depths of 0.5", 1.5", and 2.5" were used. The delamination was modeled as both and air-filled and a water-filled thin crack. Signatures were also generated for uncracked deck to establish a baseline for comparison with the cracked cases. A2 was found to be a good indicator of crack presence, as shown in Figure 5-29. The presence of a crack substantially reduces the relative amplitude of the bottom rebar return, so A2 is much smaller for a deck with an air or water filled crack than for an uncracked deck. This relationship is true for the whole examined range of cover thicknesses. Figure 5-30 shows that the relative amplitude of the top rebar return, as represented by A1, is raised by a water-filled crack, due to the higher reflection coefficient at the concrete/water interface, and is lowered by an air-filled crack, due to the lower reflection coefficient at the concrete/air interface. Figure 5-31 shows the effect of cover thickness on the difference in arrival times of the returns from the top of concrete and the top rebar, as represented by T1. A higher value of T1 is indicative of a thicker cover layer and, assuming the delamination crack is located at the plane of the top reinforcing mat, the depth of the delamination. Note that this relationship is not valid for small cover values (see Figure 5-31 graph points for 0.5" cover), due to the increased overlapping of the air/asphalt and asphalt/concrete returns.
Figure 5-29: A2 as an indicator of crack presence for various covers.
Figure 5-30: A1 as an indicator of crack filler material for a various covers
Figure 5-31: T1 as an indicator of cover and hence crack depth
The second group of cases examined the change in waveform as a waterproofing membrane failure progressed (see Appendix B). The intact membrane condition was modeled as a wet asphalt layer over dry cover and bottom concrete layers. The membrane failure was then modeled as a progressive increase in moisture in the lower layers until the entire deck was wet. Figure 5-32 shows the effect of membrane failure on the signature parameters associated with arrival times and Figure 5-33 shows the effect on parameters associated with amplitudes. There is no strong indicator of membrane failure. The membrane failure model of progressively higher moisture content has little effect on return times. The normalized return amplitudes for both top and bottom rebar fall only moderately and that of the deck bottom remains constant as concrete moisture rises. The most promising indicator from the signature parameters is the difference between A1 and A2, as shown in Figure 5-34. As the moisture content of the concrete increases, the difference between A1 and A2 increases due to higher attenuation in the lower slab layer as it gets wetter.

A better indicator could be constructed by comparing the difference in amplitude of a return from above the membrane with a return from below the membrane. Since there is no non-overlapping return from the asphalt/concrete interface, this can’t be done using this particular simple six number signature, although as discussed in Section 6.1.4, a different signature could be constructed to include more information on the asphalt/concrete boundary. The behavior of the echo from the asphalt/concrete boundary would be affected by the change in moisture of the layers, not only by the change in attenuation of the signal as it traveled through the concrete layers, but also by the amplification in reflection coefficient at the asphalt/concrete boundary due to the increased permittivity of the wetter concrete.
Figure 5-32: Effect of membrane failure on arrival time signature parameters
Figure 5-33: Effect of membrane failure on amplitude signature parameters
Figure 5-34: A1-A2 as an indicator of membrane failure
The third group of cases attempted to identify signature parameters that are indicative of chloride contamination of the asphalt and cover concrete layers. The salt content strongly affects conductivity of the pore water, as shown in Figure 5-25. Salt content in turn influences concrete conductivity as shown in Figure 5-26. The higher the moisture content of an assumed solution concentration, the higher the effect on concrete conductivity. Figure 5-35 shows the weak effect of chloride content on the signature parameters associated with arrival times. Figure 5-36 shows the effect on parameters associated with amplitudes. Either A1 or A2 could be used as an indicator of chloride content (the fact that the values for A1 and A2 are identical is an artifact of the analytic model), since the increasing attenuation due to increasing chloride is clearly represented by these parameters.

The fourth group looked at the effects of moisture and temperature both separately and together. Changing temperature alone over a wide range had no effect on return times, since the analytic model makes no variation in permittivity for changing temperatures. Signature parameters associated with return amplitudes were only modestly affected since increasing temperatures increase conductivities, causing higher attenuation and lower amplitudes. Moisture content, acting alone, was a much stronger factor. Amplitudes were reduced by half for a doubling of moisture and return times were significantly increased for higher moisture (see Appendix B). The stronger effect of moisture variation is due to the dependence of both conductivity and permittivity on moisture content. Figure 5-37 shows the effect of increasing temperature on signature parameters associated with amplitude for a normal moisture content deck. Figure 5-38 illustrates the accentuation of temperature effect for a high moisture content deck. As shown in Figure 5-39, a combined rise in temperature and moisture does not have much effect on the signature.
Figure 5-35: Effect of chloride on arrival time signature parameters
Figure 5-36: Effect of chloride on amplitude signature parameters
Figure 5-37: Effect of temperature on normalized amplitudes for medium moisture
Figure 5-38: Effect of temperature on normalized amplitudes for high moisture.
Figure 5-39: Effect of temperature & moisture on amplitude signature parameters
The fifth group looked at the effect of varying asphalt thickness on the waveform and selected signature parameters. The main effect of increasing asphalt thickness is to change the overlap behavior of the air/asphalt and asphalt/concrete returns in the early part of the waveforms. Such signal separation (see Appendix B) has clear advantages for determining paving thickness and membrane failure. A dielectric spacer, as used in radar surveys solely to determine paving thickness [Rosetta 80], could be used to achieve this separation. The main disadvantage of using a spacer is that the radar survey ceases being a noncontact method, reducing feasible operating speeds. Increasing asphalt thickness obviously increases all of the signature parameters associated with arrival times. As shown in Figure 5-40, the time parameters all increase linearly with asphalt thickness as would be expected for travel at unaltered velocities with an increased path length. Figure 5-41 shows how the increasing asphalt thickness causes a nonlinear change in amplitude signature parameters. The nonlinearity is caused by the changing overlap character of the first negative and first positive peaks as the asphalt thickness increases.
Figure 5-40: Effect of asphalt thickness on arrival time signature parameters
Figure 5-41: Effect of asphalt thickness on amplitude signature parameters
Chapter 6

Experimental design for testing program

6.1 Analytic relationships describing deck conditions

This chapter presents an experimental program design to establish how thermography and radar can be effectively used, in conjunction with other tests, to assess the condition of overlaid bridge decks. Analytic relationships describing deck condition are developed to make explicit the data requirements to be satisfied by the testing program. A test methodology is then developed so that the required information is supplied by an appropriate analytic, laboratory, or field test.

6.1.1 Deck conditions states to be determined.

The goal of deck assessment is to determine the presence, location, extent, and progress of spalling and scaling deterioration of the deck. The condition states of interest for making the assessment consist of both direct measures of delamination and scaling and indirect measures of deck conditions conducive to or associated with deterioration. Indirect measures include: chloride content, membrane performance, amount of cover, and asphalt thickness, corrosion activity, and corrosion rate. Thermography has the potential to detect: the existence of a subsurface defect, the type of the defect (delamination, debonding, scaling, or trapped water), and the depth and severity of the detected delamination. Radar has the potential to detect: the presence of delamination or scaling, excessive chloride in the cover concrete, high moisture content below the membrane, and the thicknesses of the asphalt overlay and the concrete cover.
6.1.2 Identification of experimental variables

To properly design a testing program, the significant variables must be identified so that their influences can be separated to avoid erroneous conclusions [ASCE 80]. Experimental variables may be grouped into two classes: *indicators* which indicate the presence (or absence) of a condition state, and *confounders* which confuse the interpretation of indicators or mask the presence (or absence) of a condition state.

If the variation in a test signal due to an indicator is the same order of magnitude as that due to a confounder, it may not be possible to make significant interpretation of the signal. This is equivalent to saying that the signal to noise ratio is unacceptably low. The role of an experimental variable may change between that of an indicator and that of a confounder depending on the particular test context.

6.1.3 Experimental variables for thermography

6.1.3.1 Thermal indicators

A typical thermogram is shown in Figure 6-1 [Balduman 83TRR664]. The figure uses different cross-hatching to indicate variation in infrared radiance. The display unit on the infrared system would use a grey scale with continuous contrast from black to white. Figure 6-2a is a representation of a small portion of a thermogram. As shown in Figure 6-2b, a grid may be imposed over the thermal image so that the results at a particular point \( i, j \) may be easily referenced as \( T_{i,j} \). For example, in the fictitious thermogram shown in Figure 6-2b, \( T_{5,3} \) is +2C and \( T_{9,2} \) is +1C. The spatial resolution and thermal sensitivity of the infrared system determine the mesh fineness and temperature gradation. The sensitivity of the infrared detection system is expected to be approximately 0.2C (0.36F) [Manning
80] to 0.5°C (0.9°F) [Kipp 83], but will depend on the specific detector selected. The experimental variables that are indicative of deck condition are the sensed temperature differences. With this thermal representation, the indicators for an m by n grid may be referred to as $T_{i,j}$ through $T_{m,n}$, measured in units of degrees Celsius. The range of temperatures sensed, $T_{\text{range}}$, would be found by subtracting the minimum $T_{i,j}$ from the maximum $T_{i,j}$. For example, in Figure 6-2b, $T_{\text{range}}$ is 3°C. $T_{\text{back}}$ refers to the background temperature, and is the same as the minimum $T_{i,j}$ unless there are areas cooler than the general background temperature.

### 6.1.3.2 Thermal confounders

The confounding experimental variables for thermography may be divided into deck parameters and environmental parameters. Various key words are defined to symbolize the confounding variables.

**Deck parameters:**
- **pave** = thickness of asphalt overlay (cm)
- **cover** = thickness of concrete cover (cm)
- **thick** = thickness of concrete deck (cm)
- **bargsize** = diameter of top reinforcement (cm)
- **barspace** = spacing of top reinforcement (cm)
- **emis** = emissivity of deck surface (0 to 1)
- **satP** = degree of saturation of asphalt (%) 
- **satD** = degree of saturation of deck concrete (%) 

**Environmental parameters:**
- **cloud** = amount of cloud cover (%) 
- **wind** = wind speed (km/hr) 
- **sun** = peak intensity of solar radiation (watts/m²) 
- **time** = time of day, adjusted for season and latitude (hr) 
- **ambT** = diurnal fluctuation in ambient temperature (°C) 
- **wet** = surface moisture, prevents testing (true or false)
Figure 6-1: Typical thermogram

- Warmest
- Warmer
- Medium
- Cooler
- Coolest
**Figure 6-2: Thermogram grid**

**Figure 6-2a**

**Figure 6-2b**

- Warmest, say +3°C
- Warmer, say +2°C
- Warm, say +1°C
- Background, defined as 0
6.1.4 Experimental variables for radar

6.1.4.1 Radar indicators

An idealized radar waveform, synthesized for the fifth group in Section 5.2.2, is shown in Figure 6-3. The portions of the wave representing reflections from various interfaces are labeled as follows:

0 = air/asphalt
1 = asphalt/concrete
2 = concrete/top steel
3 = concrete/bottom steel
4 = concrete/air

The amplitude and arrival of return x will be referred to as Rax and Rtx, respectively. The example returns from the air/asphalt and asphalt/concrete boundaries are well separated. To apply this notation to the general case were these returns may overlap, it is assumed that some method of separation is used, either by providing a dielectric spacer above the pavement, by using the first negative peak for air/asphalt and the first positive peak for asphalt/concrete, or by analytic wave transformation.

A desirable characteristic for the indicator/defect relationships is that they be spatially independent, requiring the analysis of a test signature from a single location. To be able to characterize the pattern of a particular waveform, it is the relative values of the peak amplitudes and the spacing between arrival times that are the variables of interest, not the absolute amplitudes and times. To account for the variation in signal energy that is transmitted into the deck, the amplitude of the air/asphalt peak, R0, will be used as a measure of signal energy and the other peak amplitudes will then be normalized by this reference value.

\[ R_{ax} = \frac{R_{ax}}{R_{0}} \]
\[ R_{0} = 1.0 \text{ (unity by definition)} \]

Likewise, the arrival times will contain an extraneous variation due to changes in
Figure 6-3: Idealized radar waveform
height of the radar source antennae above the paving surface. To eliminate this
variation, the arrival time of the air/asphalt return, R0, will be used as a base
time to which other arrival times may be consistently referenced.

\[ R_{Tx} = R_{Tx} - R_0 \text{ (nsec)} \]
\[ R_0 = 0.0 \text{ (zero by definition)} \]

The four normalized amplitudes, RA1, RA2, RA3, and RA4, and four synchronized
arrival times, RT1, RT2, RT3, and RT4, will be used as the indicator variables for
radar. The parallel to the six number signature previously discussed in Section '5.2.2
and shown in Figure 5-21 is obvious. Then eight number signature is proposed for
specification of indicator/defect relationships so that the influences of experimental
variables affecting the air/asphalt and asphalt/concrete interface returns are more
clearly separated.

**6.1.4.2 Radar confounders**

As with thermography, key words are used to symbolize the radar deck and
environmental confounding variables.

**Deck parameters:**
- **pave** = thickness of asphalt overlay (cm)
- **cover** = thickness of concrete cover (cm)
- **thick** = thickness of concrete deck (cm)
- **barsize** = diameter of top reinforcement (cm)
- **barspace** = spacing of top reinforcement (cm)
- **barloc** = location of bar relative to radar antenna (cm)
- **satP** = degree of saturation of asphalt (\%)
- **satD** = degree of saturation of deck concrete (\%)
- **saltP** = salt content of asphalt (#/cu yd)
- **saltD** = salt content of cover concrete (#/cu yd)

**Environmental parameters:**
- **temp** = average deck temperature (C)
- **wet** = surface moisture, prevents testing (true or false)
6.1.5 Analytic relationships

Having defined the deck condition states and identified the experimental variables, the analytic relations between the states and variables need to be explored. Functional forms for each condition state are proposed. The rationale for each relationship is presented and the indicators and confounders are discussed. The data required to evaluate the unknown in the proposed functions is the data that must be gathered in the test program.

6.1.5.1 Uncertainties

Uncertainty enters the analytic relationships from a variety of sources. The system used to measure the indicator signal has a limiting sensitivity which determines the degree of accuracy of the indicator value. If the test method discretely samples the continuous physical system, the samples may not be representative of the system. The relationship between the experimental variables may not be based on a well-understood causal mechanism, but may instead represent a correlation that only tends to support the state. Different test information may be contradictory. Exhaustively testing all parts of all decks to definitively establish deck condition is obviously not cost effective. The uncertainties introduced by vagueness of measurement, randomness of test sampling, and ignorance of causal mechanism must be quantified and combined in a rational manner. Techniques exist to deal with these issues, such as fuzzy sets, stochastic probability and Shaeffer-Dempster theory for combining uncertainties, but they are not further addressed in this thesis.

6.1.5.2 Thermal relationships to identify the presence of defects

Extracting information on deck condition from a thermal mapping of the deck surface temperature can be divided the three steps of first, establishing the presence
of a subsurface defect, second, identifying the defect type, and third, determining defect depth or severity. The following relationship establishes the presence of a subsurface defect as a predicate of $T_{i,j}$, $T_{range}$, and $C_1$, a coefficient between 0 and 1.

$$\text{defect}_{i,j} = \text{true if } T_{i,j} > (C_1 * T_{range})$$

The value of $C_1$ needs to be determined experimentally. As an example, if $C_1$ was taken as 0.6 in Figure 6-2b, the region outlined would be identified as a subsurface defect:

$$T_{i,j} = +2 \text{ or } +3 > (0.6 * 3) = 1.8$$

This relationship would identify the 0.03 inch air-filled crack shown in Figure 5-15 from about 9AM until 4PM.

The indicators $T_{i,j}$ and $T_{range}$ are affected by the confounders wet, emis, wind, sun, time, pave, thick, satP, satD, barsize, barspace. If moisture is present on the deck, then the difference in emissivity due to surface moisture invalidates the test [Manning 80]. The changes in deck emissivity due to texture, spalls, patches, wheel polishing, debris, stains, and paint create thermal anomalies that are not indicative of subsurface defects. A conventional video camera mounted adjacent to the infrared camera is needed to record such emissivity changes so that they are not identified as subsurface defects.

Thermography has been found to be unreliable when wind speeds are above 25 km/hr (15.5 mph) [Manning 80] to 32 km/hr (20 mph) [Kipp 83]. One hypothesis is that the wind increases the atmospheric attenuation of the reflected heat rising from the deck [Holt 79]. This hypothesis seems flawed since thermography actually detects emitted photons in the infrared band [Clemen 78], not "reflected heat". The atmosphere does absorb infrared energy in selected wavelengths [Abdel 71], but infrared systems are designed to operate within the windows of the transmission spectrum of the atmosphere [Clemen 78] and wind would not be expected to affect
this type of absorption. An alternative explanation is that the wind increases convective cooling and reduces surface temperature differences. Lab tests can be used to identify the correct mechanism. If the atmospheric attenuation hypothesis is correct, fluctuations in wind speed would correlate with detected infrared levels even when contact surface temperature measurements remained constant. If the convective cooling hypothesis is correct, fluctuations in detected infrared levels would correlate with measured surface temperature regardless of wind speed. In both cases, the maximum allowable wind speed would be expected to be a function of the other parameters determining the range of surface temperature, and should be included in the relationship for range of temperature differences.

The range of temperature differences must be above a certain minimum size if defects are to be detected. A reasonable practical minimum has been found to be 1.5C (2.7F) [Holt 79] with a 2.0C (3.6F) differential being desirable [Manning 80]. Although for actual field testing, $T_{\text{range}}$ is measured directly, it would be valuable to have a relationship giving the valid test window in terms of deck and environmental parameters. The relationship would be used to determine what decks could be tested under what weather conditions. Prediction of $T_{\text{range}}$ could be based on meteorological conditions or directly measured deck thermal gradients.

6.1.5.3 Predicting $T_{\text{range}}$ from meteorological conditions

For a set deck geometry, the expected differential is a function of the amount of solar radiation reaching the deck, the time of day, and the diurnal fluctuation in ambient temperature. The temperature differentials are expected to vary approximately sinusoidally with time of day. The $T_{\text{range}}$ values shown in Figure 5-15 vary in such a manner with a y axis offset of a constant C. The phase parameter, $\pi$, would be determined by the time of sunrise. The amplitude of the sinusoid is expected to depend additively on the range of the ambient diurnal
temperature and the amount of solar radiation reaching the deck. The parameters a, b and c represent the effect on surface temperature amplitude of ambient temperature range, peak solar radiation, and convective cooling, respectively. The ambient range is expected to weakly affect amplitude (a \leq 1). The solar radiation is expected to strongly affect amplitude (b \approx 4). If the convective cooling hypothesis for wind proves correct, an additional term is needed to show the moderate effect of convective cooling (c \approx 5/4). The expected ranges of the parameters a, b, and c could be established by use of an analytic model and then lab tests could be used to determine appropriate values.

\begin{align*}
\text{range} T &= f \{ \text{sun, cloud, time, ambT} \} \\
&= A \sin \left( \frac{2\pi}{24} \times \text{time} + \pi \right) + C \\
\text{where:} \\
A &= \text{ambT}^a + (\text{sun} \times \text{cloud})^b + \text{wind}^c.
\end{align*}

The deck conditions suitable for thermal testing are described by the allowable range of the confounders: pave, thick, satP, satD, and bars. The allowable range of environmental parameters determined by the previous relationship is expected to be indirectly proportional to slab thickness (since a thinner deck has a greater diurnal surface temperature amplitude, see Figure 5-9) and pavement thickness [Manning 85]. The effect of degree of saturation of deck materials is also expected to be indirectly proportional to the allowable environmental range since a wet slab will have a higher thermal capacity like a thicker slab. The effect of rebar size and spacing is not known but it is expected that a heavy top reinforcing mat would tend to decrease the temperature differentials by allowing rapid thermal transfer within the plane of the deck.

The data requirements for establishing the allowable range of environmental conditions subject to varying deck conditions are thus quite complex. A reasonable approach to this problem would be to first, develop an analytic model of the deck
behavior by extending the transient model previously used; second, establish experimental parameters to tune the model by conducting lab tests under controlled conditions; and third, to verify the results by field tests under various environmental conditions.

6.1.5.4 Predicting $T_{\text{range}}$ from deck thermal gradient

As shown in Figure 5-11, in the early morning, the temperature profile through the deck is essentially uniform, as the deck surface passes through a transition period from being cooler than the core of the slab during the night to being warmer during the day. As the deck begins to heat, a substantial temperature gradient is set up through the deck. When this gradient is well established, thermography can distinguish delaminated areas from sound deck. The time and power required to produce the temperature gradient determine the "window" during which thermography can be used. This suggests another approach to defining the allowable range and combination of environmental and deck configuration conditions. The dependent variable would be chosen to be the necessary temperature gradient in a sound portion of deck, $\text{gradT}_{\text{uncracked}}$. The expected range of temperatures due to defects could then be expressed as a function of defect characteristics and the uncracked temperature gradient:

$$T_{\text{range}} = f \{ \text{gradT}_{\text{uncracked}}, \text{depth}, \text{severity} \}$$

An analytic model that could predict the temperature distribution in a sound deck as a function of atmospheric conditions, deck dimensions, and material properties, could be used to explore the conditions under which the necessary temperature gradient would be ensured. Programs have been developed to calculate the distribution of temperature in bridges [Emerson 73Hirst]. However, the phenomenon of interest was not surface temperature fluctuation, but the structural effects of solar heating. To evaluate thermal loading for bridge design, the
maximum life temperature range, and the shape and size of the temperature gradients which occur through the depth of the structure are needed. These programs assume one dimensional heat flow through the bridge deck and therefore could not be used to investigate the thermal effects of properties that vary in the plane of the deck such as reinforcement size and spacing, and size and shape of damaged areas. In addition, true surface temperatures are not calculated for overlaid decks. Although these programs are not immediately appropriate for use in analytic tests for thermal behavior of defects, it may be possible to modify them for this use.

Ideally, a field instrument could be developed that, calibrated for deck configuration, would sense the deck's response to atmospheric conditions and give a positive reading when the desired thermal gradient had been established. Perhaps the simplest way of implementing this kind of in situ gradient measurement would be to core an area of the deck, verify the soundness of the location, and install thermocouples to sense the subsurface and surface temperatures, and produce a positive reading (a "green light") to indicate when thermographic testing may begin. Such an approach would have the desirable characteristic of being a direct measure of the fundamental parameter allowing valid thermographic readings, the near-surface deck temperature gradient. The instrument, as described here, has the undesirable features of requiring at least one core (more if the first location is not sound concrete), and needing to be installed in advance of the survey, but perhaps these negative features could be eliminated or minimized. The idea would be to use such instruments to ensure thermal surveys were made during the allowable testing "window".
6.1.5.5 Thermal relationships to identify the type of defects

The relationships described above result in the detection of a subsurface defect, but not identification of the defect type. The following relationships are proposed to distinguish between delaminations, debonding, trapped water, and scaling.

As discussed in Section 5.1.2, the thermal image of a debond and a delamination are expected to differ in pattern, since the flexibility of the asphalt allows contact within the outline of a debond, while the rigidity of the concrete prevents such contact within the outline of a delamination. In addition, since the debond occurs closer to the surface than the delamination, a stronger anomaly would be expected, although this has not been reported in the literature. The identification of a "marbled" anomaly could be processed digitally by determining if prominent isotherms outlined various regions inside the anomaly in a nonconcentric pattern. Figure 6-4 illustrates the expected differences in outline, pattern and shape with hypothetical delaminated and debonded thermograms.

Characteristics are not known for thermal anomalies due to trapped water, scaling, or membrane failure. The presence of a significant amount of trapped water would be expected to make an area cooler by day and warmer by night than solid deck. Since the volumetric heat of water is approximately twice that of concrete (see Table 5-I), more solar energy would be required to raise the temperature of the water, and the heat would be retained longer. If the heavy microcracking of scaling resulted in concrete with a significantly increased moisture content a similar thermal response is expected. As discussed in Section 5.1.2, moisture increases due to membrane failure are not expected to be large enough to leave a recognizable thermal signature, but this should be explored in the testing program. Preliminary field tests are needed to determine how water, scaling, and membrane failure affect the thermal image.
Figure 6-4: Typical thermograms expected for delamination and debonding
6.1.5.6 Thermal relationships to identify delamination depth and severity

The relative strength of the thermal anomalies is expected to be indicative of the depth or severity of the detected defects. In the transient analyses, delamination depth was inversely proportional to the strength of the anomaly (see Figure 5-18), and effective delamination width (severity) was directly proportional to the strength of the anomaly (see Figure 5-15). From Figures 5-15 and 18, it is not apparent how to discriminate between depth and severity using only thermograms from a single survey time. As can be seen in Figure 5-18, the time that a delamination is the same surface temperature as the solid deck is affected by delamination depth. Figure 5-15 shows no width effect, so thermograms taken at different times might be able to discriminate depth from severity. A combination of thermography and radar could be used to determine depth and severity. If the delamination depth is assumed to be equal to the cover, found from a radar measurement, then the strength of the thermal anomaly could be used to evaluate the severity of the delamination, assuming independent linear relationships between severity and anomaly strength, and depth and anomaly strength:

\[ \text{severity} = m_1 \times T_{i,j} + b_1 \]
\[ \text{depth} = m_2 \times T_{i,j} + b_2 \]

where the constants \( m_1, m_2, b_1, b_2 \) are to be determined experimentally. As illustrated in Figure 6-5, using noon data from Figures 5-15 and 18, the equations for these two lines can then be combined to form the planar equation describing the relationship of severity, depth, and anomaly strength.

6.1.5.7 Radar relationships to identify delamination

The presence of a delamination in the analytic model caused a large reduction in the amplitude of returns from below the delamination, as seen in Figure 5-29. Note that A2 in Figure 5-29, and RA3 defined in Section 6.1.4.1 are both
**Figure 6-5:** Delamination strength/severity/depth relationship
normalized measures of the return amplitude from the bottom rebar. If the amplitude of the bottom rebar echo is small, then it is likely that a delamination is present:

\[
\text{Delam1} = f \{ \text{RA3} \} \\
\text{if} \ (\ \text{RA3} < C1) \\
\text{then} \ (\text{Delam} = \text{true})
\]

where \( C1 \) = an experimentally determined constant.

A second possible mechanism for detecting delaminations is the presence of a direct reflection from a water-filled crack [Alongi 82]. An additional return was not generated in the analytic model, but a wider water-filled layer could create such a return. Lab tests could establish the feasibility of detecting water-filled delaminations in this manner.

\[
\text{Delam2} = f \{ \text{R_cr} \} \\
\text{if} \ (\text{R_cr} = \text{true}) \\
\text{then} \ (\text{Delam} = \text{true})
\]

where \( \text{R_cr} \) = presence of signal reflected from crack, not expected to be observed unless crack is filled with water.

A third possible way of detecting delaminations is based on the distortion of the waveform returned from the upper concrete region. The number of zero crossings in this portion of the waveform has been observed to be reduced if a delamination is present [Chung 84].

\[
\text{Delam3} = f \{ \text{zero_xings} \} \\
\text{if} \ (\text{zero_xings} < C1) \\
\text{then} \ (\text{Delam} = \text{true})
\]

where \( \text{zero_xings} \) = number of zero crossings in the waveform between two reliably detectable reference peaks \( P1 \) and \( P2 \).

The value of \( C1 \) and the identity of \( P1 \) and \( P2 \) are to be determined experimentally.
6.1.5.8 Radar relationships to identify scaling

Scaling causes the concrete just below the asphalt to break down into a matrix of sand and coarse aggregate which may or may not contain water. If water is present, the dielectric constant of the scaled concrete increases and there will be a strong reflection at the asphalt/concrete interface. If air is present the dielectric constant decreases and again a strong reflection is produced [Chung 84].

In addition, scaling reduces the strength of returns from below the asphalt/concrete interface. Scaling causes extreme amounts of microcracking in the cover concrete. Since cracked concrete attenuates radar signals more than sound concrete, high attenuation in the cover concrete layer may be indicative of scaling. As mentioned in Section 5.2.1.1, if it is assumed that scaled concrete has high chloride levels, this will also cause high attenuation [Ulriksen 82].

\[
\text{Scale1} = f \{ \text{RA1, RA2} \}
\]

\[
= \text{if} \ ( \text{RA1/RA2} > \text{C1})
\]

\[
\text{then} \ (\text{Scale} = \text{true})
\]

where \( \text{C1} \) = an experimentally determined constant.

The presence of trapped water above an intact membrane or debonding of the asphalt would both generate a similar signal response. Perhaps a combination of thermography and radar could be used to discriminate between trapped water, debonding, and scaling. An empirical method has been proposed for distinguishing between scaling and debonding based on the normalized amplitudes of early peaks [Chung 84]:

\[
\text{ratio1} = \text{second negative peak} / \text{first positive peak}
\]

\[
\text{ratio2} = \text{second positive peak} / \text{first positive peak}
\]

\[
\text{Scale2} = f \{ \text{ratio1, ratio2} \}
\]

\[
= \text{if} \ (\text{ratio1 is approximately} = 1.0) \text{ and } (\text{ratio2} > 0.35)
\]

\[
\text{then} \ (\text{Scale} = \text{true})
\]

\[
\text{Debond} = f \{ \text{ratio1, ratio2} \}
\]

\[
= \text{if} \ (\text{ratio1 is approximately} = 0.7) \text{ and } (\text{ratio2} > 0.20)
\]
then (Debond = true)

Field data is needed to find typical responses for delamination and debonding so that this empirical relationship may be verified and the causal mechanism understood.

6.1.5.9 Radar relationships to identify chloride contamination

Chloride contamination increases the attenuation of the radar wave, hence reducing the amplitude of the returns as can be seen in Figure 5-36. Since chlorides diffuse into the concrete from the top, the region of high contamination and high attenuation is the cover concrete. High attenuation in the cover concrete can therefore be used to infer chloride contamination:

\[ \text{Chloride} = f \{ \text{RA2, RA4} \} \]
\[ = \text{if } (\text{RA4/RA2} > C1) \]
\[ \quad \text{then } (\text{Chloride} = \text{true}) \]

where \( C1 \) = an experimentally determined constant.

This relationship states that if the concrete/top bar return is highly attenuated while the return from the bottom of the deck is not comparably attenuated, then the cover concrete is contaminated.

6.1.5.10 Radar relationships to identify membrane failure

Traditional inspection techniques do not include a reliable method to determine whether, and to what extent, a membrane is performing as a moisture barrier. As discussed in Section 3.2.7, the electrical resistivity tests have not been satisfactory, and in fact development of nondestructive procedures to determine the effectiveness of membranes is one of the many targets of the Strategic Highway Research Program [Manning 86]. A potentially economical, reliable test method for determining the effectiveness of membranes would be a valuable product, even if it did not test membrane integrity directly but instead took the indirect approach of testing moisture content of the concrete to assess membrane performance.
If a membrane is no longer functioning as a moisture barrier, the moisture content of the deck concrete below the membrane will rise. The higher the moisture content of concrete, the higher the relative permittivity and the lower the wave velocity. A possible way of determining moisture content in the concrete deck is to assume that the concrete deck thickness does not vary significantly. If the deck thickness can be measured directly at a drain or other deck penetration, the return times from the top and bottom of the deck can then be used to find the velocity of the radar in the concrete. The dielectric permittivity can then be determined and the moisture content of the concrete found. An apparent change in concrete deck thickness can then be assumed to indicate a change in moisture content. A large increase in moisture would then be interpreted to indicate a membrane failure. This simple approach relies on the assumption of uniform concrete deck thickness.

\[
\text{Membrane} = \begin{cases} \text{true} & \text{if } \left(0.5 \times \frac{RT4 - RT1}{v_c}\right) > \text{thick} \times D1 \\ \text{false} & \end{cases}
\]

where \(v_c\) = velocity of radar wave in concrete

\[v_c = \sqrt{\frac{\varepsilon_{\text{air}}}{\varepsilon_{\text{rc}}}}\]

where \(\varepsilon_{\text{rc}}\) = relative permittivity of concrete

\(v_{\text{air}}\) = velocity of radar wave in air

\(0.3 \text{ m/nsec} = 12''/\text{nsec}\)

\(D1\) represents the variation in deck thickness that can be expected under field conditions. The change in apparent thickness due to moisture changes must be larger than the actual variation in deck thickness for this approach to be successful. Using the expected variation in \(\varepsilon_{\text{rc}}\) given in Appendix B, for a deck thickness of 8", the apparent thickness increases to 10.12" if \(\varepsilon_{\text{rc}}\) increases from a dry value of 5 to a wet value of 8. Field tests should determine the variation in in situ deck thickness to see if this approach to determining membrane performance is feasible.
If the membrane of a deck has failed, the moisture content of the deck is expected to be higher than if the membrane was intact. This increased moisture content will increase the attenuation and decrease the velocity of the radar wave in the concrete deck. In addition, the reflection at the asphalt/concrete interface would be strengthened due to the increased dielectric constant of the wetter concrete. As suggested in Section 5.2.2, membrane failure could be identified by unusually high attenuation of returns from below the membrane:

\[
\text{Membrane} = f \{ \text{RA1, RA2} \}
\]

\[
= \text{if} \ (\text{RA2/RA1 < C1})
\]

\[
\text{then} \ (\text{Membrane} = \text{true})
\]

where \( C1 \) = an experimentally determined constant.

6.1.5.11 Radar relationships to measure asphalt thickness

Measurement of the asphalt cover is straightforward if the arrival times of the returns from the top and bottom of the asphalt layer are observable and if the dielectric constant of the paving is known. If the asphalt layer is thin, the two returns may overlap so that a thickness computation cannot be made. In this case, a dielectric spacer, such as a box of sand, is needed between the antenna and the top of the pavement [Rosetta 80].

If separately identifiable returns are available from the top and bottom of the asphalt, and if the dielectric constant of the asphalt is known, the thickness of the asphalt can be determined by the following:

\[
Pave = f \{ \text{RT1, } \varepsilon_{ra} \}
\]

\[
= 0.5(\text{RT} \times \text{v}_a)
\]

where \( \text{v}_a \) = velocity of radar wave in asphalt

\[
\text{v}_a = \sqrt{\frac{\varepsilon_{ra}}{\varepsilon_{rc}}}
\]

\( \varepsilon_{rc} \) = relative permittivity of asphalt

\( \varepsilon_{ra} \) = relative permittivity of asphalt

\( \text{v}_{air} \) = velocity of radar wave in air

\( \text{v}_{air} = 0.3 \text{ m/nsec} = 12''/\text{nsec} \)
The relative dielectric permittivity of the asphalt can be determined from a flat plate measurement [Chung 84]. The returns will be sufficiently separated if the asphalt thickness is at least half a wavelength.

\[ Pave \geq 0.5 \text{ pulse} \ast v_a \]

For example:

assuming pulse = 1 nsec
\[ Pave \geq 106 \text{ mm (4.2") for } \epsilon_{rc} = 2 \]
\[ 75 \text{ mm (3.0") for } \epsilon_{rc} = 4 \]

assuming pulse = 1.5 nsec
\[ Pave \geq 159 \text{ mm (6.4") for } \epsilon_{rc} = 2 \]
\[ 113 \text{ mm (4.5") for } \epsilon_{rc} = 4 \]

A study [Chung 84] using a 0.9 nsec pulse width and an average survey speed of 4.3 km/hr (2.5 mph) found an asphalt thickness of 60 mm (2.36") adequate to reliably separate the returns. Another study [Rosetta 80] using a pulse width of 1 nsec and operating speeds to 25.8 km/hr (15 mph) found it necessary to use a dielectric spacer to keep R1 from being completely masked by the transmit pulse.

The analytic model can be used to determine if the specific radar equipment to be used is expected to produce signals with unmasked and separate returns. Lab tests can then verify the chosen radar equipment is capable of return separation and hence measuring the desired range of asphalt thicknesses without requiring a dielectric spacer.

If the asphalt thickness can be accurately determined, it would be possible to remove most of the paving without disturbing the membrane [Rosetta 80]. A substantial savings would be realized if intact membranes could be left in place instead of replacing the membrane at the end of the pavement's useful life of about 15 years [Manning 82].
6.1.5.12 Radar relationships to measure concrete cover

Measurement of the concrete cover is similar to the measurement of the asphalt thickness. The arrival times of returns for top of concrete and top reinforcing and the dielectric constant of the concrete are the necessary parameters to calculate the thickness.

Thickness of concrete cover can be determined by:

\[
\text{Cover} = f \{ \text{RT1, RT2, } \varepsilon_{rc} \} \\
= 0.5(\text{RT2} - \text{RT1}) \times v_c
\]

where \( v_c \) = velocity of radar wave in the cover concrete
\[
= 0.3 / \varepsilon_{rc}^{1/2}
\]
\( \varepsilon_{rc} \) = relative permittivity of the cover concrete

Reinforcing configuration and variation in \( \varepsilon_{rc} \) are factors which complicate the measurement of cover.

Bar location, size, and spacing will alter the return time from the top reinforcing:

\[
\text{RT2} = f \{ \text{barloc, barsize, barspace} \}
\]

The effect of these bar parameters on the radar signal can be explored using the analytic model and the expected behavior can then be verified in lab tests.

The dielectric constant for concrete varies with moisture content (see Appendix B). This causes variation in wave velocity with an apparent change in cover thickness. For example:

assuming cover = 1.5\" = 0.0381m, and medium \( \varepsilon_{rc} \) = 6

\[
\text{RT2} - \text{RT1} = 0.62 \text{ nsec}
\]

if dry \( \varepsilon_{rc} \) = 5 then apparent cover = 1.37"

if wet \( \varepsilon_{rc} \) = 8 then apparent cover = 1.73"

Field tests should be used to determine variation in permittivity of in situ concrete to find if this variation must be taken into account to obtain cover measurements with the desired degree of accuracy.
6.1.6 Combining test results to make deck condition assessment

The desired end product of the research project is the development of techniques and methodologies for reliable deck assessment in a form useful to the participating states. To achieve this goal, the condition state data determined by thermography and radar must be combined with data available from other tests to arrive at an overall assessment of deck condition in the form needed for the responsible agency's decision making process. The overall assessment model to be developed, in order to be broadly useful, should be able to be modified to accommodate individual state practices and conditions.

The results of any single test in isolation are not expected to be sufficient to assess the deck's condition with the necessary completeness and reliability. For example, the thermography and radar analytic relationships previously presented implicitly assumed a single defect at any one location. The concurrence of multiple defects at one location would require combinations of relationships from various test methods to identify the multiple defects. A complete assessment is assumed to be based on a multi-test survey using complementary techniques, each providing addition information on the deck. Within this context, it is important to recognize the usefulness of simple and inexpensive methods that may not be able to directly detect defects, but can identify anomalous areas for further consideration and testing with other methods. Where test information overlaps, the results of one method can be used to cross check another and improve the reliability of the overall assessment. A successful deck survey is thus not dependent on the unanimous success of all tests.

The multi-test survey would be composed of an appropriate mix of traditional testing techniques (bridge inventory data, visual inspection, chain drag, coring, chloride samples, pachometer, electric potentials, and resistance testing) and non-
traditional methods (thermography and radar). One of the objectives of the field testing program is to determine the optimum mix of tests. The tests used on a particular deck must be tailored to yield the information needed in the decision process. If a general condition survey is being performed, percentages of deck with various defects may be an appropriate level of detail. If a deck reconstruction was to be undertaken, an accurate mapping of defect extent is needed. If prediction of future condition is needed, the various factors contributing to rate of deterioration must also be mapped.

As an example of the possible use of several tests to reach a complete condition assessment consider the following fictitious case:

- A thermographic survey indicates a deck with few delaminations.
- A radar survey identifies areas of anomalous deck thickness that are suspected to indicate high moisture below the membrane.
- Cores show that the apparent deck thickening is indeed due to a failed membrane.
- Chloride tests indicate contamination of much of the cover concrete.
- Electric potentials indicate active corrosion in a high proportion of the deck.

This scenario postulates a deck in currently serviceable condition but with a high potential for rapid deterioration. With this type of early warning information, perhaps corrective measures such as cathodic protection could be implemented before deck deterioration progressed to the point where massive repair or replacement was required.

Since the different test methods generate different forms of data, the question arises as to what is the best representational form to use for the final deck assessment. A mapping of the defect types and extents onto a scale plan of the deck
seems the most appropriate. Such a plan would be comparable to the presentation now used for deck engineering and construction drawings. It would provide an easily assimilated, compact, and convenient summary of the pertinent deck condition information. Figure 6-6 shows the use of such a map to summarize infrared and visual survey data and to recommend repairs.

To be able to interpret the data from the various tests and produce a map, information having different degrees of spatial continuity must be combined. The actual deck condition is of course spatially continuous. Some of the test methods, such as thermography and chain drag, produce spatially continuous results. However, most of the test methods produce data at discrete points. A grid of points is usually tested, and then the individual point results must be merged to create a continuous mapping. Based on the physical continuity of the deck, the discrete points of known deterioration can be aggregated into regions for mapping by grouping adjacent like points. A region growing prototype has been successfully implemented for deck assessment based on a single test method [Smit 86]. The spatial reasoning required to combine several test methods is further complicated by the different degrees of resolution and non-coincident grids that must be merged to form a single map.

6.2 Test methodology for testing program

The overall objective of the testing program is to evaluate the advanced sensory techniques of radar and thermography, in conjunction with traditional test methods, for obtaining more informative and reliable data regarding bridge deck condition. To meet this general objective, the capabilities and limitations of thermography and radar to detect different defect types must be explored. The allowable test window, especially for thermography, must be determined for a
Figure 6-8: Scale plan summary of deck assessment

- CADD section of Dan Ryan viaduct showing results of inspection and analyses.
- CADD-Generated base map
- Delaminations identified from infrared scanning
- Visual inspection of deck underside
- Visual inspection of deck underside and recommended full depth repair
- Delaminations identified from infrared scanning and recommended partial depth repair
- Recommended partial and full depth repair
variety of asphalt overlaid deck configurations under various weather conditions. Improved signal interpretation, especially for radar, must be developed for rapid, accurate defect detection. A consistent set of field data using new and traditional test methods must be gathered on a representative deck sampling to be able to determine how to best combine test results to make a complete assessment of a deck's condition. To achieve these objectives, the information needs identified in Section 6.1 must be satisfied by the test program.

6.2.1 Methods of acquiring data: analytic, lab, and field tests

Tests to understand and predict the interaction of the experimental variables may be done analytically, in the laboratory, or in the field. Each test type has advantages and disadvantages. A well designed test program uses the most appropriate type for the particular data acquisition task.

Using an analytic model to explore the interaction of experimental variables is usually the least expensive test type. Analytic studies can be performed in an office environment with few resource requirements beyond adequate computational tools. The drawback is that the studied phenomenon must be sufficiently well understood so that an analytic model may be built which captures the desired behavior. An iterative approach can be taken to building a satisfactory analytic model. A crude model is first proposed and then repeatedly corrected and refined based on lab and field data until a final model is developed that can predict actual physical behavior with the desired accuracy.

Laboratory studies involve physical testing of model specimens in simulated environments. Lab tests are usually more expensive than analytic tests and less expensive than field testing. The advantage of lab testing is the control that the experimenter can exercise over the specimen and environment. The drawback to lab
testing is the difficulty of correctly modeling the phenomenon of interest. The controlled environment makes lab studies well suited for examining fundamental behavior and initial screening of signal response to various conditions.

Exposure plot studies, an intermediate test type between lab and field, use large scale specimens under outdoor exposure conditions. Exposure plot studies are frequently used for corrosion research because of the difficulty of simulating the service environment in the lab [Manning 86]. This type of test is not expected to be used in this research program. When the control of a lab setting is not required, all tests will be conducted in the field to most directly test the performance of inspection techniques on in-service decks. During the course of the testing program, it may become appropriate to include exposure plot studies for evaluating certain environmental effects, for example diurnal temperature fluctuation for thermographic studies.

Field studies involve observing or monitoring the performance of full scale structures in the service environment. Field tests are used to evaluate the performance, validate the applicability, and perform calibration studies of the test methods in an in-service environment. Field testing for this project will be of two types: preliminary field tests and the main field study effort. Preliminary field testing will be performed to: establish in situ defect characteristics, gather data for analytic model development, verify assumptions used in site selection criteria, perform shakedown testing of field instrumentation and data acquisition, and verify that data reduction techniques are appropriate. The main field testing program will gather extensive data using traditional inspection methods, thermography, and radar on the same test areas of a cross-section of the deck population. The capabilities, limitations, and rough costs of each technique are to be established.

After the field testing, the data analysis phase establishes the significance of
the results, determines interpretation methods, and establishes the relationship between signal data and deck condition. The end product of the data analysis stage will be an implementation package of operational procedures for condition testing and data interpretation to arrive at a reliable, complete deck assessment.

6.2.2 Proposed tests

The following list groups the proposed tests by type of test environment and by sensory method. A brief statement of the data to be gathered by the test and how this meets an information need previously identified is given for each of the proposed tests.

6.2.2.1 Analytic tests

- Thermography

1. Asphalt overlay: as discussed in Section 5.1.2, the properties of the overlay will influence both amount of heat absorbed and heat transmission to the underlying concrete deck. Section 6.1.5.2 identifies pavement thickness as a confounder. Since no analytic studies have yet been done to evaluate the thermal effect of paving, it is suggested that the model be extended to explore its effects. The asphalt parameters to be examined are thickness, thermal conductivity, volumetric heat, and surface emissivity.

2. Top reinforcing bar size and spacing: Section 6.1.5.2 also identifies rebar size and spacing as confounders. Again, since their effects have not yet been explored, model extensions are suggested.

3. Allowable weather window: Sections 6.1.5.3 and 6.1.5.4 discuss two ways of predicting the ranges and combinations of deck and environmental parameters that allow valid thermal readings to be made. Analytic modeling would determine the feasibility of each approach and identify the more promising option for further development.

4. Modeling defects: the only defect type which has been explored
analytically is delamination (see section 5.1.4). Section 6.1.5.5 describes the need to develop and test modeling of debonds, trapped water, scaling, and membrane failure.

5. Delamination depth and severity: Section 6.1.5.6 proposes a planar relationship for thermal anomaly strength to delamination depth and severity. Analytic tests are needed to verify this relationship.

• Radar

1. Bar parameters: Reinforcing size, spacing, and relative location are all identified as confounders in Section 6.1.4.2, and as mentioned in Section 6.1.5.12, are especially bothersome for cover measurement. These bar parameters affect the radar waveform in several ways: First, the radar antenna receives reflected waves from a wide area of the deck, not from a narrow point. Reflections from more than one bar are included in the returns from the top and bottom steel mats. Second, the position of the antenna relative to the reinforcing effects the response, since a quicker and stronger return will be generated if the antenna is directly over a bar than if it were midway between bars. Finally, larger bars will intersect more of the radar wave path and will return a stronger reflection. With minor modifications, the existing analytic model for synthesizing radar responses can be used to study the effects of various bar parameters on the signal. A possible way of making the signal interpretation independent of antenna position relative to bar location is to combine several waveforms so that variations in reading position would be smoothed out [Smit 86]. The effects of multi-bar returns due to antenna angle width and amplitude enhancement due to bar size are both expected to be predictable in a fairly simple manner, so that signal interpretation can take these parameters into account.

2: Scaling: no tests have yet been run to specifically model scaling damage, although as mentioned in Section 6.1.5.8, salient features of scaling may be the increased moisture and chloride contents. The effect of heavy microcracking, along with the other expected characteristics should be studied to determine the typical effect of scaling on the waveform.
6.2.2.2 Laboratory tests

- Thermography

1. Allowable weather window: After analytic tests have defined the more promising of the approaches described in Sections 6.1.5.3 and 6.1.5.4, lab tests would be used to establish the influence of individual deck configuration and environmental parameters under controlled conditions.

2. Signal response to modeled defects: Tests are needed to determine typical thermal signatures of various defect types so delamination, debonding, trapped water, membrane failure, and scaling may be distinguished (see Section 6.1.5.5).

3. Delamination depth and severity: Lab tests are needed to verify the interpretive model proposed in Section 6.1.5.6.

- Radar

1. Flat plate test: The flat plate test is a straightforward, direct method for measuring the dielectric constant [Chung 84] of an exposed material. The test consists of the following steps: 1) Position the radar antenna directly over and aluminum plate placed on the ground. 2) Record the waveform and note the amplitude of the peak returned by the flat plate, A1. 3) Keeping the antenna height the same as for the flat plate case, record the waveform returned from an area of good pavement and note the amplitude of the peak returned by the asphalt surface, A2. 4) Calculate the reflection coefficient: \( r = -A2/A1 \). 5) Calculate the asphalt dielectric constant: \( \epsilon_{ra} = \left\{ \frac{1-r}{1+r} \right\}^2 \). As was discussed in the section on measuring asphalt thickness, the dielectric constant of the asphalt must be known to accurately measure the paving thickness from the radar signal. It is suggested that flat plate tests be done in the lab to determine the dielectric constants for the lab sample materials, and to verify radar equipment setup for a simple test type.

2. Bar parameters: The results of the analytic tests on bar parameter effects should be verified in a lab environment where various sets of known rebar size, spacing, and relative location values can be tested without introducing the other deck configuration variables such as deck thickness that would be
expected to occur with in-service decks with differing reinforcement patterns.

3. Defect waveforms: It is clearly desirable to know how various deck damage affects the waveform without the introduction of mixed forms of damage or confounding variables. Lab tests could be used to evaluate the suitability of the relationships proposed in Sections 6.1.5.7 to 6.1.5.10 to identify defects introduced under controlled conditions.

4. Temperature, moisture content, and attenuation: Attenuation of the radar wave within the deck affects the sensed waveform in three ways [Steinway 81]: 1) The amplitude of the signal is reduced relative to the interference level, so the power of the test equipment may be insufficient for defect detection. 2) The pulse is lengthened, due to distortion of the spectrum of the radar return since higher frequencies are attenuated more than lower frequencies. This pulse lengthening distorts the returned waveform. 3) The peak-to-peak amplitude of the received waveform is reduced, making more difficult any interpretation based on return amplitudes. Little information is available on attenuation in concrete at short-pulse radar frequencies. Variation of relative dielectric constant and dielectric conductivity for different temperatures and moisture contents is not available [Steinway 81]. Conductivity, and hence attenuation, increase with increasing temperature and increases more strongly with increasing moisture content. As discussed in Section 5.2.2, if temperature and moisture levels are too high, the attenuation of the radar signal may be excessive, and defect detection will not be possible. The objective of this lab test is to establish allowable test ranges of the environmental variables of deck temperature, paving moisture content, and concrete moisture content. A single test slab sample of sound concrete with asphalt overlay, would be needed, with an intact membrane so moisture content of concrete and asphalt can be varied independently. Metal reflector plates inserted at asphalt/concrete interface would allow collection of radar return with only asphalt layer attenuation.
6.2.2.3 Field tests

- Preliminary field data acquisition

1. Thermal effect of wind: Moderate to high wind speeds are known to degrade thermal images, but it is not clear if the causal mechanism is atmospheric attenuation of the thermal signal or convective cooling damping out the thermal signal. Section 6.1.5.2 cites the need for experimental verification of wind effects. Direct measurement of surface temperature and wind speeds are needed in addition to the associated thermograms. If thermogram intensity fluctuations correlated with wind speed fluctuations, and atmospheric mechanism would be verified.

2. Thermal signatures of defect types: Since thermal signatures of various defect types are to be produced by the analytic model and explored in lab tests, field data on defect thermal behavior is required for validation of proposed analytic and lab defect models. For example, to determine the physical characteristics and thermal behavior of an in situ delamination, the defect would be located nondestructively, a time history of its surface temperature, its radiant temperature, the ambient temperature, and the meteorological changes would be recorded, and cores would be taken to find the defect structure and composition. Time lapse photography would be a useful technique for recording the infrared time-history.

3. Radar signatures of defect types: Similarly, radar waveforms are required for various defect types for validation of analytic and lab defect models.

- Full scale field test program

1. Multi-test survey: Section 6.1.6 describes the need for conducting multiple tests on the same deck areas so that a consistent data set is available to determine the relative reliability of the various methods. The tests done in this survey will include traditional techniques (bridge inventory data, visual inspection, chain drag, coring, chloride samples, pachometer, electric potentials,
resistivity tests) and innovative methods (thermography and radar). The testing will of course be much more exhaustive than in an actual condition survey in order to generate a dense data set. The deck sites sampled must be representative of the population to which the end product of the research program will be applied.

2. Data acquisition during repair process: To be able to evaluate the performance of the tests, true deck condition must be established. Data on directly observable deck condition can be gathered during the repair or replacement process.

6.2.3 Simulating defects for laboratory testing

Laboratory testing of model specimens requires the production of slab samples with controlled defects. To obtain valid results, the specimens must properly model actual defects. As an example of potential problems in constructing specimens, a previous study found that the slab size chosen (4 ft x 4 ft) was too small, due to edge effects, to yield valid thermographic results under lab conditions. Non-uniform heating from the quartz lamp radiation sources, excessively "busy" samples due size and quantity of simulated defects, and small sample size coupled with sharp sample temperature contrast with the background, all caused difficulties in conducting and interpreting the experiments [Joyce 84]. Various methods for simulating slab defects of interest are presented below.

- **Scaling**: A previous study [Manning 82], simulated scaling by placing coarse sand to a thickness of ~10mm on top of the concrete prior to application of the tack coat and paving. Shallow scaling could be simulated by washing the plastic concrete surface with water to expose the aggregates to the desired depth [Westover 84].

- **Debonding**: A method of simulating debonding, which was found to be successful in a previous study [Manning 82], consisted of attaching circular and triangular masks to the concrete surface before application of an asphalt emulsion tack coat. After curing of the tack coat, the masks were removed, and a thin layer of powdered talc was spread on
the masked areas to act as a bond breaker, prior to application of the paving course.

- Chloride contamination: Contamination of the cover concrete could be simulated by laminating concrete with known chloride content [Westover 84], placing a top layer that is deliberately contaminated in the plastic state, or by rapid impregnation of concrete with chlorides. Rapid chloride diffusion can be accomplished by driving ion migration with the application of an imposed electric field in a manner similar to that used for rapid measurement of chloride permeability of concrete [Whiting 81].

- Delamination: One thesis [Westover 84] suggests a wide variety of untried methods for simulating delaminations in test slabs: inserting two sheets of unbonded material (paper, waxed paper, or cellophane); casting in a flat plastic air bag, embedding an ice sheet; inserting a low melting point wax and then heating to create a void; casting in a cold joint with sand or talc as a bond breaker; and finally actually inducing corrosion cracking by applying an electric potential [Federally 82]. A previous study [Joyce 84] simulated plane delaminations, curved delamination between concrete, and curved delamination between concrete and steel by removal of inserts after the concrete was set. A plane delamination was formed by inserting one or two stainless steel plates at various orientations and pulling them out after the slab was set. A curved delamination between concrete was formed by inserting a wooden cone and the replacing it with a slightly smaller concrete cone. Curved delaminations between concrete and steel were formed by rotating the rebars to debond them and removing some bars and replacing them with smaller bars.

6.2.4 Equipment requirements for thermal tests

The primary equipment requirement for infrared testing is of course the infrared thermography system itself. Although military equipment is able to detect close range temperature differences of less than 0.01°C, easily obtained commercial systems have a sensitivity of 0.1°C to 0.2°C [Abdel 71]. A commercial system with full gray scale setting of two isotherm units and a minimum thermal resolution of 0.2°C has been cited as appropriate for deck surveys [Manning 82]. Inclusion of a
short wavelength end filter to remove direct solar radiation, eliminates the direct
effect of shadows and glare, and limits sensing to the desired temperature
differences. [Joyce 84]. A van mounted system with a conventional video camera in
tandem with the infrared camera and a data acquisition system allowing digital
processing of infrared and video data is obviously advantageous. Such systems are
available for commercial surveys [Kunz 85] and it may be possible to arrange access
to such equipment for research use. the availability of prototype survey equipment
from FHWA should also be investigated.

Other measurement instruments required include:

- Precision radiation thermometer for measuring emitted radiation.

- A light meter for the infrared portion of the spectrum to measure
  incident radiation.

- Digital contact thermometers for accurate tracking of surface and
  subsurface temperatures.

- An anemometer for measuring wind speed

- Other basic meteorological instrumentation to record ambient
temperature, barometric pressure, and humidity.

Laboratory tests require an artificial radiation source such as quartz infrared
lamps. Their illuminance may measured with a visible light meter. The lamps must
be turned off while making measurements, since infrared is present in their
radiation spectrum [Joyce 84].

Field tests will require a method of making reference marks. Aluminum paint
on the pavement will decrease the emissivity and appear cooler in the infrared as
well as appearing brightly in the visible light image. Iron rods laid on the pavement
will also function as both infrared and visible markers [Joyce 84].
6.2.5 Equipment requirements for radar tests

As with thermography, the major equipment need is for the radar equipment itself. Again, the possibility of arranging access to commercial equipment and the availability of FHWA prototypes should be investigated. Radar systems suitable for use on concrete bridge decks are available from [Joyce 84]:

1. Penetradar Corporation, Incorporated, Niagara Falls, New York
2. Geophysical Systems, Incorporated, Hudson, New Hampshire, and

Commercial GPR systems use single wide band antenna for both transmit and receive. There is a trade-off between higher frequency which gives greater resolution, and the power required for non-contact penetration of the pavement. The most appropriate antenna type for the deck survey application must be identified.

The radar equipment package generally consists of [Joyce 84]:

1. A power supply
2. A transmitter which emits a microwave pulse
3. A transmit-receive selector which prevents output pulses from entering and damaging the receiver
4. A receiver which samples and converts the reflected radar wave to a low audio range signal for recording, processing, and display
5. A recording system generally made up of an analog tape recorder for permanent record and optional digital input, and a facsimile recorder for preliminary field interpretation and control of data quality and a channel monitor oscilloscope.

It is useful to have a manual event marker for identifying particular points in the survey record. In processing the data, a spiking filter may be used for analog convolution of waveform in order to straighten out wavefront. An inexpensive and
convenient form for the permanent record recorder is to use a videocassette recorder with stereo channels. A video image is recorded of the survey surface, one audio channel is used for recording the radar monitor output, and the other channel records the radar transmit pulse trigger signal. Since the survey equipment is van mounted, a tachometer with digital readout is useful to aid the driver in maintaining a constant inspection speed [Joyce 84].

Other equipment required for radar tests include:

- A probe to directly measure dielectric constants and conductivities. Applied Microwaves in Kansas manufactures an instrument for field measurement of dielectric constant and electric conductivity for material in direct contact with probe.

- A dielectric spacer [Rosetta 80]

- An instrument to measure asphalt and concrete in situ moisture contents.

- A thermometer to measure deck temperature

6.2.6 Field tests

The objective of the field tests is to evaluate the capabilities of thermography and radar, in conjunction with other tests, for assessment of bridge deck condition. The data gathered from each field site will be a complete record of the results of each test type performed on the same deck area under similar conditions. The true deck condition will be established by direct observation during the repair process or by coring for those decks not requiring repair. This data can then be analyzed to establish the strengths, limitations, and reliability of each test.

All test methods are to be investigated under documented conditions at full scale in service test sites. A grid will be established for all tests. The significant environmental conditions (surface temperature, ambient temperature in sun and
shade, time of day, solar radiation, recent rainfall, barometric pressure, wind
velocity, and humidity) will be measured and recorded. A fifth wheel will be used on
test vehicles to record distance from a set grid reference. The standoff distance for
radar antenna should be recorded, perhaps using sensors similar to pavement
profiling equipment.

Traditional testing to be included in the field tests are:

- Bridge inventory database information, past inspection reports, other
documentation normally available on the existing structure: The
purpose of obtaining these records is not only to aid in test planning
but also to establish how well these documents represent the in-service
structure.

- Visual record, above and below.

- Electric potential tests.

- Pachometer cover measurements.

- Delamination detection using both chain drag and a hammer,
  performed by a technician experienced in the technique.

- Coring at selected grid points, and at locations where some of the test
  methods identify defects or anomalous results. Information on variation
  in asphalt paving thickness and concrete deck thickness is required.
  Since ultrasound not practical [Manning 82], full thickness cores will
  be used.

When performing thermographic surveys, a conventional video camera should
be used to record the same field of view as infrared camera. Calibration areas are
required to ensure consistent readings for several passes or survey periods.
Thermocouples will be installed at several locations in the deck and on the deck
surface to record contact surface and subsurface temperatures.

When recording radar waveforms for grid, consideration will be given to using
two orientations of antenna, both standard alignment of E-field parallel to direction
of travel, and 90 degree rotation so that E-field is at right angles to direction of travel. Use of orthogonal polarization may give additional information for directional reflections from rebars and joints [Chung 84].

In addition to the exhaustive testing described above, consideration should be given to including a series of tests conducted on a selected bridge on a regular schedule, say monthly, to assess ability of the various techniques to monitor defect growth and identify limitations imposed by normal highway environment [Joyce 84].

No matter what method is used to record test data, whether direct hand recording, strip chart, or magnetic tape, it is essential to begin data analysis during testing. The reason for this is to establish the validity of the testing instrumentation, prior to the full collection and final reduction of the data. If the data appear suspect, then a full examination of the field equipment may find instrumentation faults to be corrected, or additional instrumentation requirements that must be implemented to properly measure the variables of interest [ASCE 80]. Data may be invalid due to unsuitable weather and deck conditions, equipment failures, or improper operation. To confirm a valid data set for radar it would be appropriate [Chung 84] to synthesize expected waveforms for specific deck configuration, and to verify surface returns for asphalt thickness and dielectric constant so that any great discrepancies between measured and expected waveforms may be resolved before large amounts of data are acquired. Similar data quality assurance procedures should be applied for other test types.
6.3 Field test site selection

The objective of site selection is to assure the sampled decks are representative of the deck population to which the survey techniques developed by the research program will be applied. Site selection requires the identification of the significant variables, and the selection of a range of sites that represent those variables. To develop a rationale for describing the types of decks which should be tested, and the number of decks which should be included in the field program, it is necessary to characterize the deck population to be sampled. After the deck design types and exposure variables are known, the criteria for site selection can be established, and a test matrix proposed.

6.3.1 Characteristics of the deck population to be sampled

The decks to be field tested under this program will be selected from the concrete bridge decks in the five participating New England states. The following summary of the deck design practices and general characteristics of the deck population is based on a series of meetings of Consortium participants [NESTIC 85b]. The prominent design types are used to describe the typical ranges of deck variables. The range of deck exposure variables, such as age, salting, and traffic is also examined.

6.3.1.1 Deck design in five New England states

The type of concrete deck, protective membrane system, and wearing surface varies somewhat from state to state. There is also variation based on age of bridge, since the state specifications for decks were periodically revised to take into account changes in the state of the art of bridge design. The following is a summary of typical concrete deck designs for the existing bridge population in Maine, Massachusetts, New Hampshire, Rhode Island, and Vermont.
Historically, Maine’s concrete bridge decks have had a sequence of decking designs. During the 1930’s and 1940’s, reinforced concrete wearing surfaces were typically used. The structural deck would have a thickness of about 6” with 1-1/2” clear cover. The 3” concrete wearing surface placed over the 6” deck was composed of a normal structural mix having a moderately high 5-1/2 or 6 bag cement content. The crowning was built up by varying the thickness of the wearing surface from 3” at the gutter line to 5” at the crown. The older decks now usually have 2” of bituminous paving with no membrane over the original concrete wearing surface.

Beginning in the 1940’s, bituminous wearing surfaces were used, initially with no water barrier membranes and then later with various membranes. The typical concrete deck thickness of about 7” included 1-1/2” of cover. The 2” paving was placed in two lifts consisting of a typical road paving asphalt mix. Crowning was now done by cambering the supporting structure, so the deck and paving thicknesses did not vary from gutter to crown. Beginning in the late 1950’s, built-up hot mop membranes were installed beneath the paving. During the 1970’s reinforced concrete wearing surfaces were again constructed. The wearing course was composed of 3-1/2” of regular mix structural concrete. Current practice has returned to bituminous wearing surfaces, now used with preformed single sheet membranes.

Unlike some of the other participating states, Massachusetts does have a significant number of bare decks. Most of the interstate system was constructed during the late 1950’s and the 1960’s when the state specified exposed concrete bridge decks with no membrane. The state has a large number of decks with a heavily delaminated plane at the level of the top reinforcing mat. Factors which may contribute to this extensive delamination damage are the heavy traffic and salt exposures typical of the urban area. It is also possible that construction
practices allowed improper consolidation so that a weakened plane was formed at the top mat due to honeycombing, trapping of bleed water under bars, and differential settlement of the plastic concrete.

Massachusetts did not have well defined requirements for wearing surface treatments during the 1950's and 1960's, when the majority of the interstate system was built, so there is some variation in deck configuration. In general there is 2" clear cover of concrete which is also the wearing surface on bare decks. The total cover and wearing surface thickness is about 3" for latex modified decks. Current practice primarily uses sheet membranes below bituminous overlays with some use of latex modified concrete.

New Hampshire has used asphalt wearing surfaces with membranes since the 1940's. Sheet membranes have been in use for the past 10 to 15 years. Older bridges have a hot-mop built-up membrane. A two-lift bituminous paving, with the lower lift consisting of a voidless mix with high asphalt content, is used to get longer deck lives. Although New Hampshire is heavily salted, the deck problem is not due to chloride content alone. Concrete durability, possibly exacerbated by chloride, and overloading also contribute to the problem. Skew bridges are prevalent in New Hampshire, which can restrict possible repair choices.

Rhode Island uses asphalt overlays extensively. Approximately 90% of the states' decks have bituminous overlay wearing surfaces. The older overlays were placed without membranes. Decks that are about 10 to 25 years old have built-up membranes while those that are younger have sheet membranes.

The wearing surface treatment of Vermont's bridges is also primarily bituminous overlay with no built-up or sheet membrane, depending when the paving was installed. Various membrane types have been tried and carefully
evaluated to identify those with the best in service performance [Frascoia 77]. Vermont has generally small bridges and light traffic.

6.3.1.2 Range of deck variables

Deck variables may be grouped into two categories: those associated with the as-built deck configuration, and those associated with the service exposure of the deck. Configuration variables include: superstructure type, deck thickness, reinforcement, cover, membrane type, and wearing surface type and thickness. Exposure variables are associated with expected deck condition and include: age, climate, salting history, and traffic loading. The most common superstructure for a bridge with a concrete deck is longitudinal composite design steel beams. Continuous construction is used on multiple-span bridges while simple spans are used on 1 or 2 span structures. This superstructure type was the most commonly used during the period of interstate construction when the majority of the region's highway bridges were built. The steel members are either rolled sections or built-up girders. Composite construction, using shear connectors to establish composite action between the concrete deck and the steel beams, is also a common type of superstructure. Welded stud shear connectors are typical, although some of the older 1950's bridges have spiral reinforcing shear connectors. For decks thinner than about 8-1/2", the beam area is usually haunched to provide the 8" or 9" of concrete normally required for full composite action. The third common type of superstructure is cast-in-place reinforced concrete T-beam design. This type of construction was commonly used on bridges built in the 1930's. Precast concrete beams have also been used since the 1960's, where appropriate, but are not as common as the other types.

The structural concrete deck thickness is typically 7-1/2", with a variation between 6" and 9". Use of stay-in-place form is quite rare in the region. The deck
spans 6 to 11 feet transversely between beams, with a typical beam spacing of about 7 or 8 feet. The transverse reinforcing varies somewhat with deck span, but is typically composed of #5 bars at 6" on center, top and bottom. Older designs alternated straight and bent, or "crank", bars so that every other transverse bar from the bottom mat passed up into the top mat in the region over a beam. This configuration, which provides adequate negative moment capacity over beams with slightly less steel and more bending, is not used in post 1950's decks, which instead have similar reinforcing in both top and bottom mats. Longitudinal reinforcing is typically light temperature steel composed of #4's at 6" to 12".

The most common wearing surface treatment is bituminous paving with no, built-up, or single sheet membranes, depending on the age of the paving. Massachusetts is the only state with a significant number of bare decks. Prior to the late 1950's, membranes were not commonly used. From the late 1950's to the mid 1970's built-up membranes, composed of 2 or 3 layers of fibrous material interspersed with hot mop tar, were used. Cotton fibers were quickly superseded by fiberglass. Since the mid-seventies, preformed single sheet membranes have been standard.

The material composition of the concrete and asphalt mixes does not have major variation with respect to time and location. The structural concrete mixes have some time trend changes in cement content, water cement ratio, and air entrainment. The cement content has gradually increased from 5-1/2 bags in the 30's to the higher 7 bags common since the late 50's. At the same time the water/cement ratios has decreased from more than 0.50 to less than 0.45. Since the 1940's, air-entrainment has become standard practice. Quality control on all items has improved over time. With the exception of New Hampshire, which uses a high asphalt voidless lower lift, the asphalt overlay is a typical paving mix placed in two
lifts. The overlay thickness is 2" or 2-1/2" plus about 1/4" for the membrane. There
is a recent tendency to use 3" overlays to facilitate scarifying for repaving. Thicker
overlays can accumulate from multiple pavings of the deck.

The age distribution of the bridge population can be roughly summarized as
having half the bridges dating from the period of intense interstate construction
from the late fifties to the early seventies, with the remaining half split between
earlier and later bridges with more bridges in the earlier group. The climate in New
England is such that all the states heavily salt. The urban areas of Massachusetts
and Rhode Island are particularly heavily salted. The higher number of freeze-thaw
cycles in the southern part of the region is a more severe deck exposure than the
colder northern conditions. The bridge population in Massachusetts and Rhode
Island is subjected to much heavier traffic volume than in Maine, New Hampshire,
and Vermont. The average daily traffic for bridges in urban Massachusetts is in
the 1/4 million vehicle range, which contrasts sharply with the 5 or 6 thousand
ADT's (average daily traffic) regarded as high in Vermont. The deck exposures in
the southern and urban areas of the region are thus more severe due to climate,
traffic, and the resultant heavy salting.

6.3.1.3 Survey of existing deck population

To obtain a more quantitative characterization of the existing deck
population in the five participating states, the survey form shown in Figure 6-7 was
sent out to the Consortium technical representative from each state. The completed
forms for four states are reproduced in Figures 6-8 to 6-11. Using the information
in the survey forms and the deck population statistics in Table 1-IV the quantities
of various deck types may be obtained.
Figure 6-7: Deck population survey form

State ____________ Done by ____________ Date ____________

Percentage of all bridge decks in state that are concrete __________ %

For all following questions, give the percentage of concrete decks that have the specified characteristic.

Deck configuration

Markup the following sketches to be representative of the three most common typical concrete deck details in your state.

TYPE 1

TYPE 2

TYPE 3

Transverse deck __________ %
Longitudinal deck __________ %
Stay-in-place metal forms __________ %

Superstructure type

Non-composite steel beams or girders __________ %
Composite steel beams or girders __________ %
Cast-in-place reinforced concrete __________ %
Precast reinforced concrete beams __________ %
Steel trusses __________ %
Other __________ %

Single span __________ % average length in feet __________
Multi-span __________ % average length in feet __________
Continuous spans (as opposed to simple spans) __________ %
Figure 6-8: Deck population survey results: Maine

State Maine Done by Everett Swann Date 2/10/36

Percentage of all bridge decks in state that are concrete 95%

For all following questions, give the percentage of concrete decks that have the specified characteristic.

Deck configuration

Markup the following sketches to be representative of the three most common typical concrete deck details in your state.

Type 1

Built-up Membrane

1/2" wide

1 1/2" wide

1" wide

Type 2

No Membrane

1/2" wide

6" wide

1" wide

Type 3

3" 3/4" corr. w.s.

5" 3/4" corr. w.s.

5 1/8" wide

Membrane

1/2" wide

6" 62" wide

1" wide

Transverse deck 98%
Longitudinal deck 2%
Stay-in-place metal forms 1%
Average span in feet
Some built w. 1' 6" since 1936

Superstructure type:

Non-composite steel beams or girders 25%
Composite steel beams or girders 75%
Cast-in-place reinforced concrete 30%
Precast reinforced concrete beams 0%
Steel trusses 0%
Other 0%

Single span 60% 60% average length in feet
Multi span 60% 200% average length in feet
Continuous spans (as opposed to simple spans) 30%
Figure 6-9: Deck population survey results: New Hampshire

State New Hampshire Done by Gaylor Finchmore Date August 19, 1986

Percentage of all bridge decks in state that are concrete 95 %

For all following questions, give the percentage of concrete decks that have the specified characteristic.

Deck configuration

Markup the following sketches to be representative of the three most common typical concrete deck details in your state.

Type 0

```
Concrete
1/2" curb
Post 1975
```

Type 1

```
Concrete
2 1/4" curb
Post 1975
```

Type 2

```
Membrane waterproofing
1/2" curb
Post 1967
```

Type 3

```
Performed Membrane
1" curb
Post 1980
```

Transverse deck 95 %
Longitudinal deck 6 %
Stay-in-place metal forms 0 %

Superstructure type

- Non-composite steel beams or girders 40 %
- Composite steel beams or girders 20 %
- Cast-in-place reinforced concrete 2 %
- Precast reinforced concrete beams 1 %
- Steel trusses 13 %
- Other

Single span 75 %
Multi span 25 %

Average length in feet:
- 50 for single span
- 150 for multi span

Continuous spans (as opposed to simple spans) 15 %
Figure 6-10: Deck population survey results: Rhode Island

State Rhode Island  Done by  Date 8/28/82

Percentage of all bridge decks in state that are concrete 95%

For all following questions, give the percentage of concrete decks that have the specified characteristic.

Deck configuration

Markup the following sketches to be representative of the three most common typical concrete deck details in your state.

Type 27

3-ply Membrane

Type 3607 Mopped waterproofing

transverse deck 98%
longitudinal deck 2%

stay-in-place metal forms %

Superstructure type

non-composite steel beams or girders 60%
composite steel beams or girders 40%
cast-in-place reinforced concrete 20%
prefabricated reinforced concrete 10%
other 5%

single span 80'
multi span 100'

continuous spans (as opposed to simple spans) 7%

Note: All above data is approximate. The exact data is not readily available.
Figure 6-11: Deck population survey results: Vermont

State Vermont Done by Warren Trapp Date 8/2/76

Percentage of all bridge decks in state that are concrete ___ %

For all following questions, give the percentage of concrete decks that have the specified characteristic.

Deck configuration

Markup the following sketches to be representative of the three most common typical concrete deck details in your state.

Transverse deck 100 % average span in feet 80
Longitudinal deck ___ % average span in feet ___

Stay-in-place metal forms ___ %

Superstructure type

Non-composite steel beams or girders 40 %
Composite steel beams or girders 30 %
Cast-in-place reinforced concrete 20 %
Precast reinforced concrete beams ___ %
Steel trusses ___ %
Other ___ %

Single span 40 % 80 average length in feet
Multi span 60 % 80 average length in feet

Continuous spans (as opposed to simple spans) 15 %
6.3.2 Criteria for site selection

There are certain criteria which can be set for any bridge to be included in the field tests. The deck must be accessible both above and below so tests may be performed. The wearing surface must be bituminous paving since this is the focus of the research program. Stay-in-place forms, longitudinal deck spans, and continuous spanning superstructures are excluded due to their small representation in the deck population. Superstructures are restricted to the most common types: non-composite steel, composite steel, and cast-in-place concrete beams.

It is important to distinguish the effects which are unrelated to deck condition, but do affect the performance of the measurement techniques (confounders), from the effects which are related to condition, and are associated with various expected types and levels of deterioration (indicators). This distinction must be made so that signal variations can be interpreted as having to do with factors related or unrelated to deck condition, thus gathering the data required to be able to infer deck condition from signal responses for a range of deck types and test circumstances. To accomplish this goal, a number of field sites can be selected so that the deck condition is held constant while the signal variables are changed. This means that a number of field sites, considered "reference" sites, would be selected to represent the non-condition related variables, but which otherwise can be assumed to be in good condition. These sites will establish a baseline for the expected performance of radar and thermography under various design and environmental conditions.

Confounder variables can be broken into two groups: those which may vary at a particular bridge site over a relatively short period of time and those which remain constant for a deck unless maintenance, repair, or reconstruction takes place. The first dynamic type consists of: weather (cloud, wind, rain, humidity,
temperature), time, and moisture content of overlay and deck. The second static type consists of: surface emissivity, wearing surface type and thickness, membrane type, deck composition (concrete mix, cover, reinforcing, deck thickness) and superstructure type. Reference sites are specified to have a representative range of the static deck variables. Tests performed on one or more of these decks at several times would establish signal response to a range of dynamic variables.

Surface emissivity varies due to: surface texture, spalls, patches, polishing, debris, and stains. One of the reference sites should be selected to have a wearing surface with varying emissivity while the other reference sites would have a relatively uniform surface. If the reference site had an area relatively free of changes as well as an area with well-defined variations, the effect of changes in surface emissivity could be easily determined without confusion with the effects of other variables.

Since all tested decks will have bituminous overlays, the asphalt mix and thickness and water barrier treatment are the significant variations in overlay. Four reference sites would be needed to represent the following common wearing surface treatments:

1. A deck with 1-1/2 to 2" asphalt wearing surface with no membrane.

2. A deck with 1-1/2 to 2" asphalt wearing surface with a built-up membrane.

3. A deck with 2 to 2-1/2" asphalt wearing surface with a sheet membrane.

4. A deck with 2-1/2 to 3" asphalt wearing surface with a sheet membrane.

Since wearing surface treatment is primarily determined by bridge age, the four reference sites chosen above would also represent a variation in the deck
composition variables. Since concrete mix, cover, reinforcing, and deck thickness all are assumed to have modest variability within the sampled deck population, additional reference sites are not included to control separately for these parameters. These four reference sites will all have non-composite steel beam superstructures while two additional reference sites will be selected to assess the effects of composite steel and cast-in-place concrete superstructures. If, as expected, type of superstructure has little affect on the signals of interest, these reference sites will be the only structures tested that are not non-composite steel construction.

The matrix shown in Figure 6-12 shows the proposed groupings of reference sites and expected deck condition states. The 16 cases cited provide a cross-section of the major population and condition characteristics.

Scheduling of the testing program must be carefully coordinated with deck repair and rehabilitation work, so that the availability of equipment and manpower may be matched to the contractor's schedule.
**Figure 6-12: Field Test Site Selection Matrix**

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Chapter 7

Context of the research program

The body of this thesis has been concerned with the underlying physical basis and assessment potentials of the advanced sensory techniques of radar and thermography. A research program to evaluate the performance of specific techniques must be placed in a broader context if the results are to eventually lead to the desired goal of practically implementable improvements in the task of concrete bridge deck assessment. The proposed testing program does indeed have such a broader context, provided by the integrated research program for deck condition assessment currently planned by the New England Surface Transportation Infrastructure Consortium. The objectives and approach of this overall research framework are presented [NESTIC 86] to place the test program in its appropriate context and to outline the steps to ensure development of practically useful results.

7.1 Research objectives

Given the problem statement of Chapter 1 and the current practice survey and critique of Chapter 3, the following objectives are identified for a research program to address the deck assessment needs of the New England states.

1. Improved quality of condition data.

2. Effective and efficient inspection techniques.

3. Effective use of condition data.

4. A final product suitable for practical implementation.
These goals are based on a definition of the problems as seen by the participating agencies [NESTIC 85a, NESTIC85b, NESTIC85c] and relies on their identification of a desired solution.

The first objective of the program is to develop and implement techniques for obtaining more accurate and reliable information regarding bridge deck condition. This includes not only improved and expanded data acquisition but also data interpretation. New measurement techniques are needed to obtain better and more informative data. Interpretive techniques are needed for extracting significant signals from raw data, for combining results from different types of tests, and for incorporating a priori information on deck design, environment, and maintenance history into the condition interpretation. The condition information provided by these techniques should be sufficient to: select repair candidate decks; determine appropriate repair approaches; and predict future condition if no repairs are required.

The second objective of the program is to insure that the techniques developed can be efficiently and economically implemented. This will ensure that: a large number of decks can be evaluated each year; the evaluations will be affordable; and that the benefits of the improved information will exceed the cost of obtaining it.

The third objective is to develop analysis and decision-making tools and techniques which help to make the most of the condition information that can be provided. Such tools will help to define the type of condition survey that is most economical and appropriate for a given deck. They will also provide a means for determining and evaluating the various maintenance, repair, and replacement options.

The final objective is to organize the research results into a bridge deck
maintenance management system as depicted in Figure 7-1. This system will integrate the following elements: existing knowledge of the deck and its environment; knowledge of the available inspection and condition assessment techniques and their expeditious application; data generated by a condition survey; understanding of the physics of bridge deck deterioration; and understanding of the various maintenance options and their various economic and social impacts. These elements will be packaged to provide the user with a set of tools (manuals, reports, and computer programs) to assist in making the various decisions required for assessing condition and specifying maintenance.

7.2 Research approach

The objectives defined above can be effectively achieved through a program composed of three phases. The phases are associated with the achievement of specific objectives. The phased approach thus provides a means for monitoring the course and achievement of the program, and for systematically organizing the research effort. Figure 7-2 shows the interactions of the various phases and identifies the satisfaction of the four objectives.

The focus of Phase I is the test program to evaluate the capability of advanced sensory techniques for bridge deck condition assessment, with specific emphasis on ground penetrating radar and infrared thermography. This is the area on which this thesis has concentrated. The field tests will highlight a range of deck designs, environmental exposures, and test conditions. The results of these field tests will be documented in a detailed report describing the strengths, limitations, and capabilities of ground penetrating radar and infrared thermography when used to evaluate bridge deck conditions. Phase I will thus meet the first objective of improving the quality of condition data.
Figure 7-1: Bridge deck condition assessment & maintenance management system

- Input
- USER
- condition assessment, best repair or replace options
- Presentation and Analysis of Repair/Replace Options (life cycle costing)
- interpreted condition data
- Analysis and Interpretation of Survey Data
- raw survey data
- Conduct Survey
- Detailed Specification of Condition Survey
- general survey specifications
- Determination of Appropriate Condition Survey
- Input Bridge Deck Data (age, ADT, environment, design, maintenance, etc.)

- V
- V
Figure 7-2: Deck condition research program: a phased approach
Phase II consists of two parts to be pursued concurrently. The first part, the development of techniques and procedures for implementing the techniques evaluated in Phase I in the context of an overall inspection program, is required to satisfy the second objective of practical utilization of the advanced inspection methods. This work will focus on test equipment, test methodology, use of different tests to provide complementary data, interpretation of test data, and use of data from sources other than physical tests. The second part, the development of approaches for proposing and evaluating maintenance, repair, and replacement alternatives utilizing condition data, is necessary to meet the third objective of using the assessment data fully. These approaches will identify: the kind of condition data needed; the costs and benefits of acquiring more data; the best repair strategies for a given condition assessment and the trade-offs between repair and total replacement.

Phase III focuses on the packaging of the results of the first two phases into a practical bridge deck management system. Specific tools and techniques will be developed for implementing the test techniques and analysis approaches developed during Phases I and II. The final product of Phase III will be the manuals, guides, computer programs, and documentation which can be used by each state highway agency for implementing the bridge deck management system including:
1. A tool for determining the most appropriate and cost effective survey for a given deck.

2. A guide for generating detailed specifications for a given type of survey, utilizing newly developed and existing survey techniques.

3. A tool for interpreting the results of a given survey, in the form of a detailed condition assessment.

4. A tool for determining the various maintenance, repair, rehabilitation, and replacement options based on the condition assessment, and for evaluation these options from a technical, economic, and social perspective.

Phase III thus satisfies the final objective of delivering a useful end product capable of improved bridge deck condition assessment and utilization of assessment information to better support bridge management decisions. Figure 7-1 shows the flow of information through the final bridge deck condition assessment and maintenance management system. Inventory and maintenance data are the initial input, used to determine the appropriate survey strategy. The interpreted survey results and the initial data are used to determine the best assessment, repair or replacement options. All the pertinent data is thus gathered and structured into a systematic decision making framework.

7.3 Conclusion

A systematic approach has been presented to develop methods for improved determination of the condition of asphalt covered concrete bridge decks. The approach taken builds on existing experience and technologies. Instead of attempting to produce radically new test methods, existing but immature techniques are targeted for practical development and all available data sources are to be integrated and more fully utilized. The benefits to be obtained by the proposed
research program are many. The basic achievement will be to improve bridge management decisions due to the availability of better condition assessments of the existing deck population.
Appendix A

Finite element analysis of physical thermal model of delamination

The NEKTON [Nekton 86] program was used to solve the transient conductive heat transfer expressible by the equation:

\[ \rho \frac{d}{dt} T = k \nabla^2 T + q_v \text{ in } D_{\text{f}} U D_{\text{s}} \]

where \( \rho \) = the volumetric specific heat, assumed time-independent and constant in \( D_{\text{f}} \), and time-independent and elementally-piecewise constant in \( D_{\text{s}} \)

\( T \) = temperature

\( t \) = time

\( k \) = thermal conductivity, assumed time-independent and constant in \( D_{\text{f}} \), and time-independent and elementally-piecewise constant in \( D_{\text{s}} \)

\( q_v \) = volumetric heat generation, taken to be a general function of space and time in \( D_{\text{f}} \), and time-independent and elementally-piecewise constant in \( D_{\text{s}} \)

\( D_{\text{f}} \) = fluid part of domain

\( U \) = union

\( D_{\text{s}} \) = solid part of domain

\( \frac{d}{dt} \) = first partial differential with respect to time

\( \nabla \) = the vector gradient

\[ = i \frac{dt}{dx} + j \frac{dt}{dy} + k \frac{dt}{dz}. \]

The finite element grid and input files for the example problems discussed in section 5.1.4 are given in this Appendix. The following set of units was used throughout:

\[ \text{temperature: kelvin} \]

\[ \text{length: meter} \]

\[ \text{time: hour} \]
energy: joule

Heat transfer coefficients were iteratively determined for radiant and convective losses at the top and bottom surfaces of the slab.

The incident solar radiation was modeled with a thin fluid top layer with a time dependent volumetric heat generation, \( q_v \), determined such that:

\[
q_v = \frac{q_s}{d}
\]

where \( q_s \) = parabolic approximation of incident solar radiation
\( d \) = thickness of heat generating layer.

The determination of \( q_s \) included consideration of reflectivity. The periodic variation of the ambient temperature was modeled indirectly since the program requires a time independent sink temperature at infinity. All fluid model layers include a time dependent volumetric heat generation, \( q_v \), determined such that:

\[
q_v = -\pi \cdot d \cdot \frac{d}{t_a}
\]

where \( T_a \) = diurnal ambient temperature.

This produces a solution to a constant sink problem which differs from the varying sink solution only by the difference between the constant sink and varying sink temperatures at each time point. The desired solution can thus be easily found from the computed solution:

\[
T_{\text{actual}} = T_{\text{computed}} + (T_v - T_c)
\]

where:
- \( T_{\text{actual}} \) = desired solution temperature for varying sink temperature \( T_v \)
- \( T_{\text{computed}} \) = computed solution temperature for constant sink temperature \( T_c \)

With the exception of the diurnal variation in incident solar radiation and ambient temperature, the development of the computational model was straightforward. A sinusoid was used for daily temperature variation. To allow the deck to reach steady periodicity, all cases were run for three day cycles and only the third cycle data were used.
The input files and finite element grids follow. Note that the x scale is enlarged by a factor of 5 with respect to the y axis to improve legibility. The first file gives the Fortran subroutines describing the driving forces of incident solar radiation and ambient temperature. The finite element grid and input file for each of the eight cases discussed in section 5.1.4 then follow in numeric order. The grids were checked for fineness to ensure numerical results independent of the mesh. In addition, an effort was made to use similar grids for all cases so that similar degrees of convergence to stable periodicity would be obtained after three daily cycles. The size of grid elements was chosen so that the dimensional variation in adjacent elements was not excessive.
Figure A-1: Input file for thermal driving forces

SUBROUTINE INFLOW(IEL, ISIDE, X, Y, UX, UY, T)
RETURN
END
SUBROUTINE INITCS(UX, UY, T, X, Y)
T = 22.5
RETURN
END
SUBROUTINE DRIVEF(PGRADX, FLOW, QSRC, TIME, X, Y)
real sunrise, sunset, solar, tt, a, b, c, parabola, tempsin
integer n
sunrise = 4.75
sunset = 19.25
a = -9.515
b = 228.336
c = -869.914
n = time / 24.0
tt = time - (n * 24.0)
parabola = 0.7 * (a * tt**2 + b * tt + c) * 3600.0 / 0.005
tempsin = -4.335E+06 * cos(0.2618 * (time - 9.7))
if (tt.le.sunrise) solar = 0.0
if ((tt.ge.sunrise).and.(tt.le.sunset)) solar = parabola
if (tt.ge.sunset) solar = 0.0
if (y.ge.0.0) qsdc = solar
if (y.le.0.0) qsdc = tempsin
RETURN
END
Figure A-2: Case 1 finite element grid: thick slab
Figure A-3: Case 1 input file: thick slab
Figure A-4: Case 2 finite element grid: thin slab
Figure A-5: Case 2 input file: thin slab

SET PHOM 2.200E+6
SET CONDUCT 0.000E+00
SET FINTIME 96.0
SET DT 1.0
SET ITIME 96.0
SET EN 5
SET GRID 0.01
BUILD

F <- B.C. FOR ELE 1 SIDE 2
C <- B.C. FOR ELE 1 SIDE 3
32450.00
22.50000
F <- B.C. FOR ELE 1 SIDE 4
F <- B.C. FOR ELE 2 SIDE 2
F <- B.C. FOR ELE 2 SIDE 4
F <- B.C. FOR ELE 3 SIDE 2
F <- B.C. FOR ELE 4 SIDE 2
F <- B.C. FOR ELE 4 SIDE 4
F <- B.C. FOR ELE 5 SIDE 2
F <- B.C. FOR ELE 5 SIDE 4
F <- B.C. FOR ELE 6 SIDE 2
F <- B.C. FOR ELE 6 SIDE 4
C <- B.C. FOR ELE 7 SIDE 1
F <- B.C. FOR ELE 7 SIDE 2
F <- B.C. FOR ELE 7 SIDE 4

Coordinates of Historical Point:
-1.000000E+00
-1.000000E+00
-1.000000E+00
-1.000000E+00
-1.000000E+00

Coordinates of Historical Point:
-1.000000E+00
-1.000000E+00
-1.000000E+00
-1.000000E+00
-1.000000E+00

EXIT
Figure A-8: Case 3 finite element grid: 0.01'' air-filled crack
Figure A-7: Case 3 input file: 0.01" air-filled crack
Figure A-8: Case 4 finite element grid: 0.03" air-filled crack
Figure A-9: Case 4 input file: 0.03" air-filled crack

SET PHOCP 2.000E+0
SET COND 8640.00
SET FTIME 96.0
SET DT 1.0
SET ITIME 96.0
SET ETYPE 5
SET GRID 0.10
BUILD

0.500000E+00 0.500000E+00 ← SCALE FACTORS XFAC, YFAC
-0.500000E+00 -0.400000E+00 POINT 1 ← XZERO, ZZERO
0.400000E+00 0.000000E+00 POINT 2
0.400000E+00 0.500000E+00 POINT 3
0.400000E+00 0.000000E+00 POINT 4
0.400000E+00 -0.190500E+01 POINT 5
0.400000E+00 -0.190500E+01 POINT 6
0.400000E+00 0.000000E+00 POINT 7
0.400000E+00 -0.381000E+01 POINT 8
0.400000E+00 -0.381000E+01 POINT 9
0.400000E+00 -0.381000E+01 POINT 10
0.400000E+00 -0.190500E+01 POINT 11
0.400000E+00 -0.190500E+01 POINT 12
0.400000E+00 -0.695500E+01 POINT 13
0.400000E+00 -0.445000E+01 POINT 14
0.400000E+00 -0.445000E+01 POINT 15
0.400000E+00 0.000000E+00 POINT 16
0.400000E+00 0.107950E+00 POINT 17
0.400000E+00 0.107950E+00 POINT 18
0.400000E+00 0.107950E+00 POINT 19
0.400000E+00 -0.695500E+01 POINT 20
0.400000E+00 -0.190500E+01 POINT 21
0.400000E+00 0.000000E+00 POINT 22
0.400000E+00 0.107950E+00 POINT 23
0.400000E+00 0.107950E+00 POINT 24
0.400000E+00 -0.445000E+01 POINT 25
0.400000E+00 0.445000E+01 POINT 26
0.400000E+00 -0.381000E+01 POINT 27
0.400000E+00 -0.381000E+01 POINT 28
F ← B.C. FOR ELE 1 SIDE 2 0.000000E+00
C ← B.C. FOR ELE 1 SIDE 3 32600.00 22.5000
F ← B.C. FOR ELE 1 SIDE 4
F ← B.C. FOR ELE 2 SIDE 2 0.000000E+00
F ← B.C. FOR ELE 2 SIDE 4
F ← B.C. FOR ELE 3 SIDE 2 0.000000E+00
F ← B.C. FOR ELE 3 SIDE 4
F ← B.C. FOR ELE 4 SIDE 2 0.000000E+00
F ← B.C. FOR ELE 4 SIDE 4
F ← B.C. FOR ELE 5 SIDE 2 0.000000E+00
F ← B.C. FOR ELE 5 SIDE 4 0.000000E+00
C ← B.C. FOR ELE 6 SIDE 1 30650.00
F ← B.C. FOR ELE 6 SIDE 2 0.000000E+00
F ← B.C. FOR ELE 6 SIDE 4
F ← B.C. FOR ELE 7 SIDE 2 0.000000E+00
F ← B.C. FOR ELE 7 SIDE 4 0.000000E+00
0.000000E+00 ← M

7 715.0 9960.0 0.0

HISTORY
2.000000E+02 0.000000E+00 ← Coordinates of historical Point
-1.300000E+02 -1.300000E+00 ← Coordinates of historical Point
-6.499999E+02 -6.499999E+00 ← Coordinates of historical Point
-8.165000E+02 -0.190500E+01 ← Coordinates of historical Point

EXIT
Figure A-10: Case 5 finite element grid: 0.05" air-filled crack
Figure A-11: Case 5 input file: 0.05" air-filled crack
Figure A-12: Case 6 finite element grid: 0.05" water-filled crack
Figure A-13: Case 6 input file: 0.05" water-filled crack
Figure A-14: Case 7 finite element grid: 2" cover
Figure A-15: Case 7 input file: 2" cover
Figure A-17: Case 8 input file: 2.5" cover

```
SET RMOCF 2.298E+4
SET CONDUCT 8644.80
SET FINTIME 96.0
SET DT 1.6
SET IOTIME 96.0
SET ETypeT 5
SET GRID 0.10

BUILD

T

0.50000E-01 0.50000E-01
-0.50000E-02 -0.50000E-02
0.00000E+00 0.00000E+00
6.40000E-01 0.50000E-02
0.00000E+00 0.50000E-02
0.00000E+00 -0.31750E-01
6.40000E-01 -0.31750E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -0.63500E-01
6.40000E-01 -0.63500E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -0.82500E-01
6.40000E-01 -0.82500E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -1.07500E-01
6.40000E-01 -1.07500E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -1.35000E-01
6.40000E-01 -1.35000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -1.65000E-01
6.40000E-01 -1.65000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -1.98500E-01
6.40000E-01 -1.98500E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -2.35000E-01
6.40000E-01 -2.35000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -2.75000E-01
6.40000E-01 -2.75000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -3.15000E-01
6.40000E-01 -3.15000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -3.55000E-01
6.40000E-01 -3.55000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -3.95000E-01
6.40000E-01 -3.95000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -4.35000E-01
6.40000E-01 -4.35000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
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6.40000E-01 -4.75000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -5.15000E-01
6.40000E-01 -5.15000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -5.55000E-01
6.40000E-01 -5.55000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -5.95000E-01
6.40000E-01 -5.95000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
0.00000E+00 -6.35000E-01
6.40000E-01 -6.35000E-01
6.40000E-01 0.00000E+00
0.00000E+00 0.00000E+00
C

--- SCALE FACTORS XFAC, YFAC
C

F <- B.C. FOR ELE 1 SIDE 2
0.00000E+00
22.50000
C

F <- B.C. FOR ELE 3 SIDE 2
33860.00
F <- B.C. FOR ELE 4 SIDE 2
0.00000E+00
F <- B.C. FOR ELE 5 SIDE 2
0.00000E+00
F <- B.C. FOR ELE 6 SIDE 2
0.00000E+00
F <- B.C. FOR ELE 7 SIDE 2
0.00000E+00
F <- B.C. FOR ELE 8 SIDE 2
0.00000E+00
F <- B.C. FOR ELE 9 SIDE 2
0.00000E+00

--- Coordinates of Historical Point

T

HISTORY
2.000000E-02 0.000000E+00
2.000000E-02 -1.300000E-02
2.000000E-02 -0.499999E-02
2.000000E-02 -0.165000E0
2.000000E-02 -0.198500E0
INITCOND
T = 22.5
OUTPUT
TDFP
EXIT
```
Appendix B

Appendix B Synthetic radar waveforms for bridge deck defects

The SYNTH\textsuperscript{1} [Maser 85b] program was used to generate waveforms for 27 cases of deck dielectric configurations. The basic deck geometry is shown in Figure B-1. The standard input file for this set of cases is shown in Figure B-2. To prevent variations in waveforms from change in the relative location of the antenna and reinforcing, the speed of the survey vehicle, radar pulse generation rate, and location of the transverse line was selected so that all waveforms were generated with the antenna positioned midway between both longitudinal and transverse bars. (The first nine cases were inadvertently run with the antenna position slightly off-center between transverse bars, but since all these cases are internally consistent for relative antenna location, the data is still valid.)

The values for dielectric properties were determined using the equations presented in Section 5.2.1 and the following assumptions. Uncontaminated concrete in a state of normal moisture content was assumed to have a relative

\textsuperscript{1}As stated in section 5.2.2, the code for this computer program contains a bug. The transmission coefficients $t_{12}$ and $t_{21}$ are both computed as $1 - |t_{12}|^2$. The correct values of the transmission coefficients are $t_{12} = 2Z_2 / (Z_2 + Z_1)$ and $t_{21} = 2Z_1 / (Z_1 + Z_2)$. The effect of this error is to excessively reduce the amplitude of the detected signal for each interface through which the pulse travelled. Since each additional layer compounds the error, the portion of the waveform containing signals from the lower layers of the deck have amplitudes which are much too low. All waveforms which were generated using SYNTH have this error so all amplitudes below the first interface are distorted.
Figure B-1: Deck geometry for radar cases
Figure B-2: Standard deck input parameters for radar cases

Bridge Id: 100
Bridge Id: Continue (y,n)? y
Deck Length (ft.): 9.5
Deck Width (ft.): 9.5
Thickness of Deck (in.): 7.5
Diameter of Rebars (in.): 0.5
Center to Center Spacing of Rebars (in.): 6.0
Depth to Centerline of Top Transverse Rebar (in.): 1.75
Depth to Centerline of Top Parallel Rebar (in.): 2.25
Depth to Centerline of Bottom Transverse Rebar (in.): 5.75
Depth to Centerline of Bottom Parallel Rebar (in.): 5.25
Thickness of Asphalt Overlay (in.): 2.0
Are Transverse Bars Perpendicular to the Wheel Paths (y,n)?
  if ('n') Angle of skew (degrees): y
Water/Cement Ratio Specified (decimal or 0 if not known (.45 assumed)): .45
Age of Bridge Deck (years): 20
Number of Months Since Last Inspection (months): 24
Type of Inspection that Occurred (1 - visual; 2 - corrosion potential) 1
  if (= 1) Visual Rating (number from 1 to 10): 6
  if (= 2) Percentage of Area ... Than -.35 volts (%):
  if (= 3) Percentage of Area ... was Detected (%):
    if (= 4) Percent of Samples Content Exceeded 0.033
Number of Years Since Previous Reconstruction (years): 20
Average Number of Salt Applications Per Year: 40
Freeze Thaw Environment (1 - Mild; 2 - Moderate; 3 - Severe): 2
Climate Moisture Rating (1 - Low; 2 - Medium; 3 - High): 2
Traffic Rating (1 - Low; 2 - Medium; 3 - High): 2
Speed of Survey Vehicle (mph): 12.954545
Radar Pulse Generation Rate (waveforms/sec): 4
Frequency of Radar Pulse (MHz): 900
Height of Radar Antenna (ft.): 6
Type of Radar Pulse (1 = Sinusoidal; 2 = Gaussian): 1
Number of Traverse Lines: 2
  for (i = 0; i < n_lines)
    Position of Traverse Line #i (ft.): 4.75
Number of Data Points Per Waveform: 555
Average Temperature for 24 Hours Preceding Measurement (degrees F): 68
Average Relative Humidity for 2 Week Period Preceding Measurement (%): 75
dielectric constant of 6.0 [Ulriksen 82] and a conductivity of $5.0 \times 10^{-3}$ mhos/m [Hope 85]. Assuming a volumetric concrete composition of 70% aggregates, 5% entrained air and 25% hardened cement paste (which is 20% voids) the concrete porosity was assumed to be 10% [Illston 81]. Concrete was assumed to have a moisture content of 2.5% if dry, 5% if normal, and 10% if wet. Asphalt [Harmondsworth 62] is an inherently more porous medium than concrete, with an open void structure of 10% or more. An asphalt porosity of 15% was assumed with a moisture content of 3.75% if dry, 7.5% if normal, and 15% if wet.

The range for chloride contamination, measured in #salt/cuyd, was selected to include the range encountered in a survey of in service bridge decks of 0.23 to 12.5 [Spellman 70]. The maximum chloride ion content which is allowed in fresh uncontaminated reinforced concrete which will be exposed to chloride in service is 0.15% by weight of cement [ACI 318 83]. Assuming a 5.5 bag mix, this gives 0.78#salt/cuyd, which for a 5% moisture content yields a solution conductivity of 2 mhos/m. Using Archie's law modified for unsaturated mixtures, this value cross-checks with the assumed concrete conductivity of $5.0 \times 10^{-3}$ mhos/m. The conductivity of asphalt and concrete for various chloride contents were computed using the following linear relationship:

$$\sigma_{m1} = \left( \frac{\sigma_{s1}}{\sigma_{s0}} \right) \times \sigma_{m0}$$

where:
- $\sigma_{m0}$ = conductivity of uncontaminated mixture (mhos/m)
- $\sigma_{m1}$ = conductivity of contaminated mixture (mhos/m)
- $\sigma_{s0}$ = conductivity of uncontaminated solution (mhos/m), and
- $\sigma_{s1}$ = conductivity of contaminated solution (mhos/m).

The groups of cases examined are described in Section 5.2.2. An index to the individual cases is given in Figure B-3. The specific dielectric properties used for each case and the synthesized waveforms are then presented in the indexed order.
### Figure B.3: Index to 27 radar cases

<table>
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<th>1</th>
<th>2</th>
<th>3</th>
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<tbody>
<tr>
<td><strong>Uncracked:</strong></td>
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<tr>
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<td></td>
<td></td>
</tr>
<tr>
<td>Maximum X Value (ft.)</td>
<td>9.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum Y Value (ft.)</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Y Value (ft.)</td>
<td>9.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conductivity of Asphalt (mho/m)</td>
<td>1.25e-3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relative Permittivity of Asphalt</td>
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<td></td>
</tr>
<tr>
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<td></td>
</tr>
<tr>
<td>Relative Permittivity of Concrete Layer 1</td>
<td>6.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conductivity of Crack (mho/m)</td>
<td>5.00e-3</td>
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<td></td>
</tr>
<tr>
<td>Relative Permittivity of Crack</td>
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<td></td>
</tr>
<tr>
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<td>Relative Permittivity of Concrete Layer 2</td>
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</tr>
<tr>
<td>Top Bar Cover (in.)</td>
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<td>1.5</td>
<td>2.5</td>
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</table>

<table>
<thead>
<tr>
<th></th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Water-filled Crack:</strong></td>
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<td></td>
</tr>
<tr>
<td>Minimum X Value (ft.)</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Maximum X Value (ft.)</td>
<td>9.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum Y Value (ft.)</td>
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<td></td>
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<tr>
<td>Maximum Y Value (ft.)</td>
<td>9.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conductivity of Asphalt (mho/m)</td>
<td>1.25e-3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relative Permittivity of Asphalt</td>
<td>2.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conductivity of Concrete Layer 1 (mho/m)</td>
<td>5.00e-3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relative Permittivity of Concrete Layer 1</td>
<td>6.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conductivity of Crack (mho/m)</td>
<td>8.00e-3</td>
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<td></td>
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<tr>
<td>Relative Permittivity of Crack</td>
<td>6.0</td>
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<td></td>
</tr>
<tr>
<td>Conductivity of Concrete Layer 2 (mho/m)</td>
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<td>Relative Permittivity of Concrete Layer 2</td>
<td>6.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top Bar Cover (in.)</td>
<td>0.5</td>
<td>1.5</td>
<td>2.5</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th></th>
<th>7</th>
<th>8</th>
<th>9</th>
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</thead>
<tbody>
<tr>
<td><strong>Air-filled Crack:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum X Value (ft.)</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum X Value (ft.)</td>
<td>9.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum Y Value (ft.)</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Y Value (ft.)</td>
<td>9.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conductivity of Asphalt (mho/m)</td>
<td>1.25e-3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relative Permittivity of Asphalt</td>
<td>2.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conductivity of Concrete Layer 1 (mho/m)</td>
<td>5.00e-3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relative Permittivity of Concrete Layer 1</td>
<td>6.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conductivity of Crack (mho/m)</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relative Permittivity of Crack</td>
<td>1.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conductivity of Concrete Layer 2 (mho/m)</td>
<td>5.00e-3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relative Permittivity of Concrete Layer 2</td>
<td>6.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top Bar Cover (in.)</td>
<td>0.5</td>
<td>1.5</td>
<td>2.5</td>
</tr>
</tbody>
</table>
Figure B-4: Input parameters: delamination model

**Group I: delamination model**

1. uncracked 0.5" cover  
2. uncracked 1.5" cover  
3. uncracked 2.5" cover  
4. water-filled crack 0.5" cover  
5. water-filled crack 1.5" cover  
6. water-filled crack 2.5" cover  
7. air-filled crack 0.5" cover  
8. air-filled crack 1.5" cover  
9. air-filled crack 2.5" cover  

**Group II: membrane failure model**

1. wet asphalt / dry cover / dry concrete  
2. wet asphalt / medium cover / dry concrete  
3. wet asphalt / medium cover / medium concrete  
4. wet asphalt / wet cover / medium concrete  
5. wet asphalt / wet cover / wet concrete  

**Group III: chloride contamination model**

1. 0.25#/cu yd salt in asphalt and cover  
2. 1.00#/cu yd salt in asphalt and cover  
3. 4.00#/cu yd salt in asphalt and cover  
4. 16.00#/cu yd salt in asphalt and cover  

**Group IV: combined moisture and temperature effects**

1. medium moisture: 4°C  
2. medium moisture: 21°C  
3. medium moisture: 38°C  
4. high moisture: 4°C  
5. high moisture: 21°C  
6. high moisture: 38°C  

**Group V: asphalt thickness effects**

1. 1.0" paving  
2. 2.5" paving  
3. 5.0" paving
Figure B-5: Radar waveforms: uncracked
Figure B-6: Radar waveforms: water-filled crack
Figure B-7: Radar waveforms: air-filled crack
membrane failure model

<table>
<thead>
<tr>
<th></th>
<th>wet</th>
<th>wet</th>
<th>wet</th>
<th>wet</th>
<th>wet</th>
</tr>
</thead>
<tbody>
<tr>
<td>asphalt cover</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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<tr>
<td>version</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
</tbody>
</table>

Minimum X Value (ft.) : 0.0
Maximum X Value (ft.) : 9.5
Minimum Y Value (ft.) : 0.0
Maximum Y Value (ft.) : 9.5
Cond. of Asphalt (mho/m): 5.00e-3
Rel. Per. of Asphalt : 4.0
Cond. of Conc. Layer 1 : 1.25e-3 5.00e-3 5.00e-3 2.00e-2 2.00e-2
Rel. Per. Conc. Layer 1 : 5.0 6.0 6.0 8.0 8.0
Cond. of Crack (mho/m) : 1.25e-3 5.00e-3 5.00e-3 2.00e-2 2.00e-2
Rel. Per. of Crack : 5.0 6.0 6.0 8.0 8.0
Cond. of Conc. Layer 2 : 1.25e-3 1.25e-3 5.00e-3 5.00e-3 2.00e-2
Rel. Per. Conc. Layer 2 : 5.0 5.0 6.0 6.0 8.0
Top Bar Cover (in.) : 1.5
Figure B-9: Radar waveforms; membrane failure model

Amplitude

- --- wet/dry/dry + 2.0
- --- wet/med/dry + 1.5
- --- wet/med/med + 1.0
- --- wet/wet/med + 0.5
- --- wet/wet/wet + 0.0

Time, 32 points = 1 nanosecond
chloride contamination model

all layers medium moisture

<table>
<thead>
<tr>
<th>salt/cu yd asphalt&amp;cover</th>
<th>0.25</th>
<th>1.00</th>
<th>4.00</th>
<th>16.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>version</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
</tbody>
</table>

Minimum X Value (ft.) : 0.0
Maximum X Value (ft.) : 9.5
Minimum Y Value (ft.) : 0.0
Maximum Y Value (ft.) : 9.5
Cond. of Asphalt (mho/m): 3.66e-3 1.46e-2 5.83e-2 2.33e-1
Rel. Per. of Asphalt : 2.0
Cond. of Conc. Layer 1 : 1.63e-3 6.48e-3 2.49e-2 1.04e-1
Rel. Per. Conc. Layer 1 : 6.0
Cond. of Crack (mho/m) : 1.63e-3 6.48e-3 2.49e-2 1.04e-1
Rel. Per. of Crack : 6.0
Cond. of Conc. Layer 2 : 5.00e-3
Rel. Per. Conc. Layer 2 : 6.0
Top Bar Cover (in.) : 1.5
Figure B-11: Radar waveforms: chloride contamination model
### Temperature Effects

<table>
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<tr>
<th>Moisture All Layers</th>
<th>Med</th>
<th>Med</th>
<th>Med</th>
<th>Wet</th>
<th>Wet</th>
<th>Wet</th>
</tr>
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<tbody>
<tr>
<td>Temperature C</td>
<td>4</td>
<td>21</td>
<td>38</td>
<td>4</td>
<td>21</td>
<td>38</td>
</tr>
<tr>
<td>Version</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
</tr>
</tbody>
</table>

- **Minimum X Value (ft.):** 0.0
- **Maximum X Value (ft.):** 9.5
- **Minimum Y Value (ft.):** 0.0
- **Maximum Y Value (ft.):** 9.5
- **Cond. of Asphalt (mho/m):** 6.73e-4, 1.14e-3, 1.61e-3, 2.69e-3, 4.56e-3, 6.43e-3, 4.0, 4.0, 4.0, 2.57e-2, 8.0, 8.0
- **Rel. Per. of Asphalt:** 2.0, 2.0, 2.0, 2.0, 2.0, 2.0, 2.0, 2.0, 2.0
- **Cond. of Conc. Layer 1:** 2.69e-3, 4.56e-3, 6.43e-3, 1.08e-2, 1.28e-2, 1.82e-2, 1.82e-2, 2.57e-2, 2.57e-2
- **Rel. Per. Conc. Layer 1:** 6.0, 6.0, 6.0, 6.0, 6.0, 6.0, 6.0, 6.0, 6.0
- **Cond. of Crack (mho/m):** 6.0, 6.0, 6.0, 6.0, 6.0, 6.0, 6.0, 6.0, 6.0
- **Rel. Per. of Crack:** 6.0, 6.0, 6.0, 6.0, 6.0, 6.0, 6.0, 6.0, 6.0
- **Cond. of Conc. Layer 2:** 2.69e-3, 4.56e-3, 6.43e-3, 1.08e-2, 1.82e-2, 1.82e-2, 2.57e-2, 2.57e-2, 2.57e-2
- **Rel. Per. Conc. Layer 2:** 6.0, 6.0, 6.0, 6.0, 6.0, 6.0, 6.0, 6.0, 6.0
- **Top Bar Cover (in.):** 1.5

- **Figure B-12:** Input parameters: moisture and temperature effects
Figure B-13: Radar waveforms: moisture and temperature effects
Figure B-14: Input parameters: asphalt thickness effects

<table>
<thead>
<tr>
<th>asphalt thickness effects</th>
</tr>
</thead>
<tbody>
<tr>
<td>all layers medium moisture</td>
</tr>
</tbody>
</table>

| asphalt thickness inch | 1 | 2.5 | 5.0 |

| Minimum X Value (ft.) | 0.0 |
|Maximum X Value (ft.) | 9.5 |
|Minimum Y Value (ft.) | 0.0 |
|Maximum Y Value (ft.) | 9.5 |
|Cond. of Asphalt (mho/m) | 1.25e-3 |
|Rel. Per. of Asphalt | 2.0 |
|Cond. of Conc. Layer 1 | 5.00e-3 |
|Rel. Per. Conc. Layer 1 | 6.0 |
|Cond. of Crack (mho/m) | 5.00e-3 |
|Rel. Per. of Crack | 6.0 |
|Cond. of Conc. Layer 2 | 5.00e-3 |
|Rel. Per. Conc. Layer 2 | 6.0 |
|Top Bar Cover (in.) | 1.5 |
Figure B-15: Radar waveforms: asphalt thickness effects
References

[AASHTO 76a] Manual for Bridge Maintenance
American Association of State Highway and Transportation
Officials, 1976.

Federal Highway Administration, April, 1976.

Depth to Ground-Water Table by Remote Sensing.
Journal of the Irrigation and Drainage Division, American Society
of Civil Engineers: pages 355-367, September, 1971.

[ACI 222 85] American Concrete Institute Committee 222.
Corrosion of Metals in Concrete.
Journal of the American Concrete Institute Volume 82(Number 1),
1985.

[ACI 318 83] American Concrete Institute Committee 318.
Building Code Requirements for Reinforced Concrete.
American Concrete Institute, Detroit, Michigan, 1983.

[Alongi 82] Alongi, A. V. et. al.
Concrete Evaluation by Radar Theoretical Analysis.
Transportation Research Record (Number 853): pages 31-37, 1982.

[ASCE 80] Committee on Safety of Bridges.
A Guide for the Field Testing of Bridges.
Technical Report, American Society of Civil Engineers, New
York, 1980.

Standard Test Method for Half Cell Potentials of Reinforcing
Steel in Concrete.
ASTM, 1985, pages 557-563.

Standard Test Method for Electrical Resistivity of Membrane-
Pavement Systems.
1985 Annual Book of ASTM Standards: Volume 4.03.
*Concrete Bridge Deck Deterioration and Repair.*
Technical Report Civil Engineering R83-01, Massachusetts
Institute of Technology, January, 1983.

[Bungey 82] Bungey, J. H.
*The Testing of Concrete in Structures.*

[Bungey 83] Bungey, J. H.
*Concrete,* August, 1983.

[Busa 84] Busa, G. D.
Modeling Concrete Bridge Deck Deterioration.
Civil Engineering MS Thesis.

[Cady 83] Cady, P. D. and R. E. Weyers.
Chloride Penetration and the Deterioration of Concrete Bridge
Decks.
*Cement, Concrete, and Aggregates* Volume 5(Number 2):pages 81-87, Winter, 1983.

[Cantor 78] Cantor, T., and C. Kneeter.
Radar and Acoustic Emission Applied to Study of Bridge Decks,
Suspension Cables, and Masonry Tunnel.
Bridge Design, Evaluation, and Repair.

[Chung 84] Chung, T., C. R. Carter, and D. G. Manning.
*Signature Analysis of Radar Waveforms Taken on Asphalt Covered
Bridge Decks.*
Technical Report ME-84-01, Canada Ministry of Transport and
Communications, June, 1984.

Detecting of Delamination in Bridge Decks with Infrared
Thermography.
[Clemena 85] Clemena, G. G.
Survey of Bridge Decks with Ground-Penetrating Radar: a Manual
Virginia Highway and Transportation Research Council, July,
1985.

[Collingbourne 75]
Collingbourne, R. H.
The United Kingdom Solar Radiation Network and the
Availability of Solar Radiation Data from the Meteorological
Office for Solar Energy Applications.
In Conference on U. K. Meteorological Data and Solar Energy
Applications at the Royal Institution London. International

[Cordon 66] Cordon, W. A.
Freezing and Thawing of Concrete.
American Concrete Institute, 1966.
ACI Monograph No. 3.

[CRC 72] Handbook of Chemistry and Physics, 53rd Edition

[Ede 67] Ede, A. J.
An Introduction to Heat Transfer Principles and Calculations.

[Emerson 73] Emerson, M.
The Calculation of the Distribution of Temperature in Bridges.
Technical Report LR 561, Transportation and Road Research
Laboratory, Crowthorne, Berkshire, 1973.

[Federally 82] Annual Progress Report, Category 4, Improved Materials
Utilization and Durability
Federally Coordinated Program of Research and Development in

[FHWA83 83] Manual for Maintenance Inspection of Bridges
American Association of State Highway and Transportation
Officials, 1983.

Field Test of Reinforcement Corrosion in Concrete.
In Performance of Concrete in a Marine Environment, pages
205-221. American Concrete Institute SP 65, 1980.
[Fisher 84] Fisher, J. W.
*Fatigue and Fracture in Steel Bridges.*

[Frascoia 77] Frascoia, R. I.
*Evaluation of Bridge Deck Membrane Systems and Membrane Evaluation Procedures.*


*Thermal Modelling of a Leaky Roof.*
unpublished.

[Harmondsworth 62] Harmondsworth, Engineering Road Research Laboratory.
*Bituminous Materials in Road Construction.*

[Hirst 82] Hirst, M. J. S.
Thermal Loading of Concrete Bridges.
In *International Conference on Short and Medium Span Bridges.*
Toronto, 1982.

Detecting Concrete Bridge Deck Delaminations with Infrared Thermography.

Corrosion and Electrical Impedance in Concrete.

[Illston 81] Illston, J. M., J. M. Dinwoodie, and A. A. Smith.
*Concrete, Timber and Metals.*
Use of AC Impedance Technique in Studies on Steel in Concrete
in Immersed Conditions.
*British Corrosion Journal* Volume 16(Number 2):pages 102-106,
1981.

[Joyce 84] Joyce, R. et. al.
*Rapid Non-destructive Delamination Detection.*
Technical Report FHWA/RD-84/076, Federal Highway

Effect of Water Infiltration of Penetrating Cracks on
Deterioration of Bridge Deck Slabs.
*Transportation Research Record* (Number 950), 1984.

Thermographic Investigation of a Bridge Deck.
*Public Works* :pages 70-73, September, 1983.

Evaluation of Bridge Deck Condition by the Use of Thermal
Infrared and Ground-Penetrating Radar.
In *Second International Bridge Conference, Pittsburgh, PA.* June,
1985.

[Malloy 69] Malloy, J. F.
*Thermal Insulation.*

Detecting Delaminations in Concrete Bridge Decks.
*Concrete International* :pages 34-41, November, 1980.

*Detecting Deterioration in Asphalt-Covered Bridge Decks.*
Technical Report ME-82-03, Research & Development Branch
Ontario Ministry of Transportation & Communications,
September, 1982.

Detecting Deterioration in Asphalt-Covered Bridge Decks.
*Transportation Research Record* (Number 899):pages 10-20, 1983.
Manning, D. G. and F. B. Holt.  
*The Development of D.A.R.T. (Deck Assessment by Radar and Thermography.*  

*Technical Research Area #4 Detailed Planning for Research on Bridge Component Protection.*  

*Automation of Condition and Deterioration Surveys Using Knowledge-Based Signal Processing.*  
unpublished.

Maser, K. R.  
*Intelligent Systems for the In-Situ Evaluation of Materials and Structures.*  

McNeill, J. D.  
*Electrical Conductivity of Soils and Rocks.*  

Federal Highway Administration.  
*National Bridge Inventory Database.*  
April, 1986.  
unpublished.

National Cooperative Highway Research Program.  
*Detecting Defects and Deterioration in Highway Structures.*  
[NCHRPS4 70] National Cooperative Highway Research Program.  
*Concrete Bridge Deck Durability.*  

*Durability of Concrete Bridge Decks.*  

[Nekton 86] Karniadakis, G. E.  
*Nekton Users Manual.*  

[NESTIC 85a] New England Surface Transportation Infrastructure Consortium.  

[NESTIC 85b] New England Surface Transportation Infrastructure Consortium.  
*Summary of Meeting on Bridge Decks held at MIT on December 17, 1985.*  
Technical Report, Department of Civil Engineering, Massachusetts Institute of Technology, December, 1985.

*Technologies for Concrete Bridge Deck Condition Assessment.*  
Technical Report, Department of Civil Engineering, Massachusetts Institute of Technology, May, 1986.

[Neville 73] Neville, A. M.  
*Properties of Concrete.*  

[Park 80] Park, S. H.  
*Bridge Inspection and Structural Analysis: Handbook of Bridge Inspection.*  

[Ramo 84] Ramo, S., J. R. Whinnery, and T. Van Duzer.  
[Rosetta 80] Rosetta, J. V.
unpublished.

[Seymour 85] Seymour, W. J.
Bridge Management Systems.
Master's thesis, Massachusetts Institute of Technology, December, 85.
Civil Engineering Engineer's Thesis.

[Smit 86] Smit, D. S.
Knowledge-Based Interpretation of Elevated Roadway Condition Surveys.
Master's thesis, Massachusetts Institute of Technology, February, 86.
Electrical Engineering and Computer Science Master's Thesis.

Chlorides and Bridge Deck Deterioration.

[Stark 71] Stark, D.
Studies of the Relationships Among Crack Patterns, Cover Over Reinforcing Steel, and Development of Surface Spalls in Bridge Decks.

*Locating Voids Beneath Pavement Using Pulsed Electromagnetic Waves.*

[Threlkeld 70] Threlkeld, J. L.

[Treadway 79] Treadway, K. W. J.
Durability of Steel in Concrete.
In *Corrosion of Steel Reinforcements in Concrete Construction.*
[Ulriksen 82] Ulriksen, C. P. F.
Application of Impulse Radar to Civil Engineering.

[USDOT 79] Federal Highway Administration.
Recording and Coding Guide for the Structure Inventory and
Appraisal of the Nation's Bridges.
Technical Report FHWA, United States Department of

[USDOT 85a] United States Department of Transportation, Federal Highway
Administration.
Secretary of Transportation: Report to Congress: The Status of the
Nation's Highways: Condition and Performance.

[USDOT 85b] United States Department of Transportation, Federal Highway
Administration.
Highway Bridge Replacement and Rehabilitation Program: Sixth
Annual Report to Congress.

[Vanzetti 72] Vanzetti, R.
Practical Applications of Infrared Techniques.

[Verbeck 75] Verbeck, G. J.
Mechanisms of Corrosion of Steel in Concrete.
In Corrosion of Metals in Concrete. American Concrete Institute
SP 49-3, 1975.

[Westover 84] Westover, P. L.
Highway Bridge Inspection and Non-Destructive Evaluation of
Concrete Bridge Decks.
Master's thesis, Massachusetts Institute of Technology, June, 84.
Civil Engineering MS Thesis.

[Whiting 81] Whiting, D.
Rapid Measurement of the Chloride Permeability of Concrete.
Public Roads Volume 45(Number 3):pages 101-112, December,
1981.
XADAR, Inc.
Technical Application Note 3: Data Interpretation Techniques.

unpublished.