CENTER FOR
PORTLAND CEMENT CONCRETE
PAVEMENT TECHNOLOGY

Development of In-Situ Detection Methods for Materials-Related Distress (MRD) in Concrete Pavements

Phase 1 Report
July 2003

Department of Civil, Construction, and Environmental Engineering
IOWA STATE UNIVERSITY

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16. Abstract

The purpose of this research was to summarize existing nondestructive test methods that have the potential to be used to detect materials-related distress (MRD) in concrete pavements. The various nondestructive test methods were then subjected to selection criteria that helped to reduce the size of the list so that specific techniques could be investigated in more detail. The main test methods that were determined to be applicable to this study included two stress-wave propagation techniques (impact-echo and spectral analysis of surface waves techniques), infrared thermography, ground penetrating radar (GPR), and visual inspection. The GPR technique was selected for a preliminary round of "proof of concept" trials.

GPR surveys were carried out over a variety of portland cement concrete pavements for this study using two different systems. One of the systems was a state-of-the-art GPR system that allowed data to be collected at highway speeds. The other system was a less sophisticated system that was commercially available. Surveys conducted with both sets of equipment have produced test results capable of identifying subsurface distress in two of the three sites that exhibited internal cracking due to MRD. Both systems failed to detect distress in a single pavement that exhibited extensive cracking. Both systems correctly indicated that the control pavement exhibited negligible evidence of distress. The initial positive results presented here indicate that a more thorough study (incorporating refinements to the system, data collection, and analysis) is needed. Improvements in the results will be dependent upon defining the optimum number and arrangement of GPR antennas to detect the most common problems in Iowa pavements. In addition, refining high-frequency antenna response characteristics will be a crucial step toward providing an optimum GPR system for detecting materials-related distress.

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DEVELOPMENT OF IN-SITU DETECTION METHODS FOR MATERIALS-RELATED DISTRESS (MRD) IN CONCRETE PAVEMENTS

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The authors would like to thank the Iowa Department of Transportation (Iowa DOT) and the Federal Highway Administration for sponsoring this research. Special thanks to Kevin Jones at the Iowa DOT for his assistance in providing the traffic control and cores for this research project. Without his help, plus the help of field personnel from the Iowa DOT, this project could not have been completed. Special thanks to John Whited (Iowa DOT) and Ray Mandli and Paul Oldenburg (Mandli Communications, Inc.) for their help on the pavement management related issues present in this report. Finally, the authors would like to thank the remaining members of the research team—Allan Dudding, Kejin Wang, and Igor Beresnev—for their participation on this project.

INTRODUCTION

For many years, researchers have been interested in developing techniques that could be used to evaluate the “quality” of concrete pavements. Hansen (*1*) elucidated his thoughts as follows:

> This investigation was undertaken to determine the possibility of isolating from a study of cores some of the factors responsible for surface and progressive scaling of concrete in pavements exposed to frost action. Before it is possible to evaluate the influence of cement and aggregate in the scaling of the concrete it is necessary to determine the quality of the workmanship with which the concrete was prepared and placed.

Many pavement and materials engineers will look at a distressed pavement and come up with different theories as to the cause of the distress. The concrete industry continues to search for the knowledge of how materials and construction techniques can be combined to enhance the performance of portland cement concrete (PCC) pavements. One such effort is the work of Michigan Tech University to develop a systematic approach to detection, analysis, and treatment of materials-related distress (MRD). This approach has been thoroughly documented in a recent report (*2*). The following definition of MRD will be used in this study (*2*):

> In general, MRD refers to concrete failures that are a direct result of the properties of the materials and its interaction with the environment to which it is exposed. In this sense, these failures are differentiated from others that may be most closely associated with inadequate design for the traffic and environmental loading or the use of improper practices during pavement construction.

There is a need for a method (or methods) to evaluate and quantify the amount of distress or deterioration present in existing pavement slabs. Since MRD might begin at the base of the slab, evaluation is difficult and commonly subjective. Currently, petrographic examination of core sections is the most common technique that is used. However, it is also time consuming, semi-quantitative (or opinion based in most instances), and dependent on observations of surface features. Hence, the purpose of this phase of the research project was as follows:

- Conduct a literature survey on nondestructive test (NDT) methods that could potentially be used to make in-situ detection of materials related distress.
- Conduct brief field studies of promising techniques in an effort to gain experience with instrumentation, methodology, and interpretation prior to conducting the “proof of concept” studies in Phase 2.

The scope of this project was limited to concrete pavement sites located in Iowa. The goal of this project is to identify NDT methods that could be used to supplement the existing analysis techniques outlined by Michigan Tech University for detection and analysis of MRD. In addition, an attempt was made to clarify and help document the research and development that will be needed to make NDT methods more effective in helping to diagnose MRD.

OVERVIEW OF FIELD DATA COLLECTION ACTIVITIES

Prior to making a diagnosis of materials-related distress, one must locate the distress. This sounds simple; however, it is not. Pavements stretch for miles and it takes a keen eye to recognize when a discolored area on a slab is really related to internal distress. Extensive work from the Long Term Pavement Performance (LTPP) project has clearly indicated that the severity of the observed distress is difficult to describe quantitatively, especially when different individuals conduct the surveys (*3*). Recent work from the Canadian LTPP tended to be in general agreement with this fact (*4*). Also, there are instances where extensive distress is present but it is not directly evident from observations of the surface of the slab. Similarly, surface cracking can appear to be severe but sometimes it is not critical because it is only of a surficial nature and fails to penetrate deep into the slab. The purpose of this section is to present an overview of the fieldwork that is needed to locate, document, and extract pavement cores for subsequent analysis for MRD. A brief summary of how the Iowa Department of Transportation (Iowa DOT) conducts such activities is also given.

A typical field distress survey would proceed as shown in Figure 1 (*2*). Although each state may alter the basic components illustrated in the diagram to better suit their personnel and the level of technology that is available, it is still apparent that the process will consume many hours of labor for at least several individuals. After the survey data have been collected, the flow chart in Figure 2 can be used to assess the potential that the distress is materials related. If the distress is materials related then pavement cores can be extracted for further analysis. The recommended coring pattern is detailed in Figure 3. Petrographic analysis via ASTM C 856 (*5*) is the most common technique that is used to document MRD.

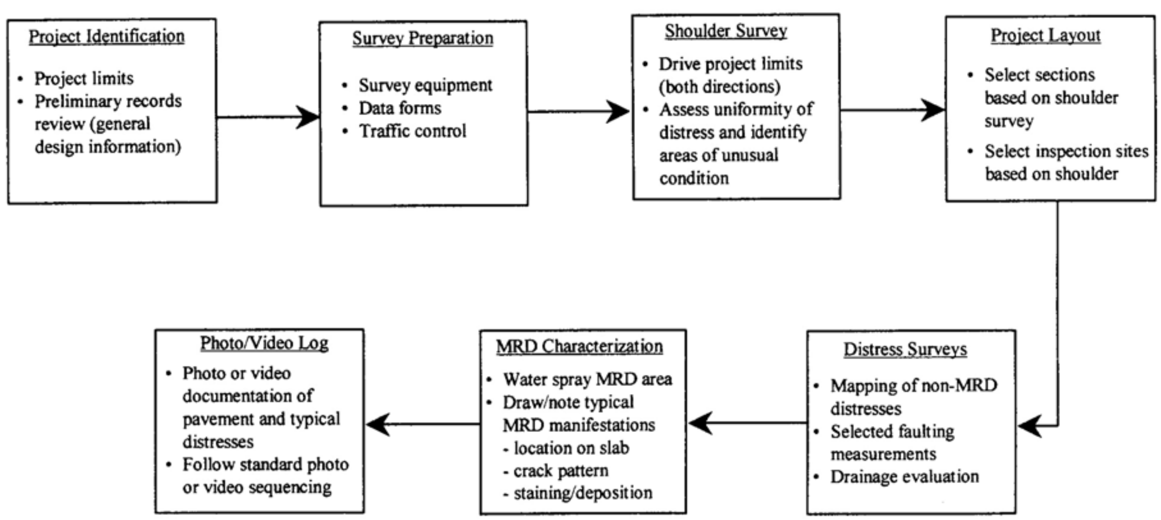


Figure 1. Flow of Field Data Collection Activities (from ref. *2*)

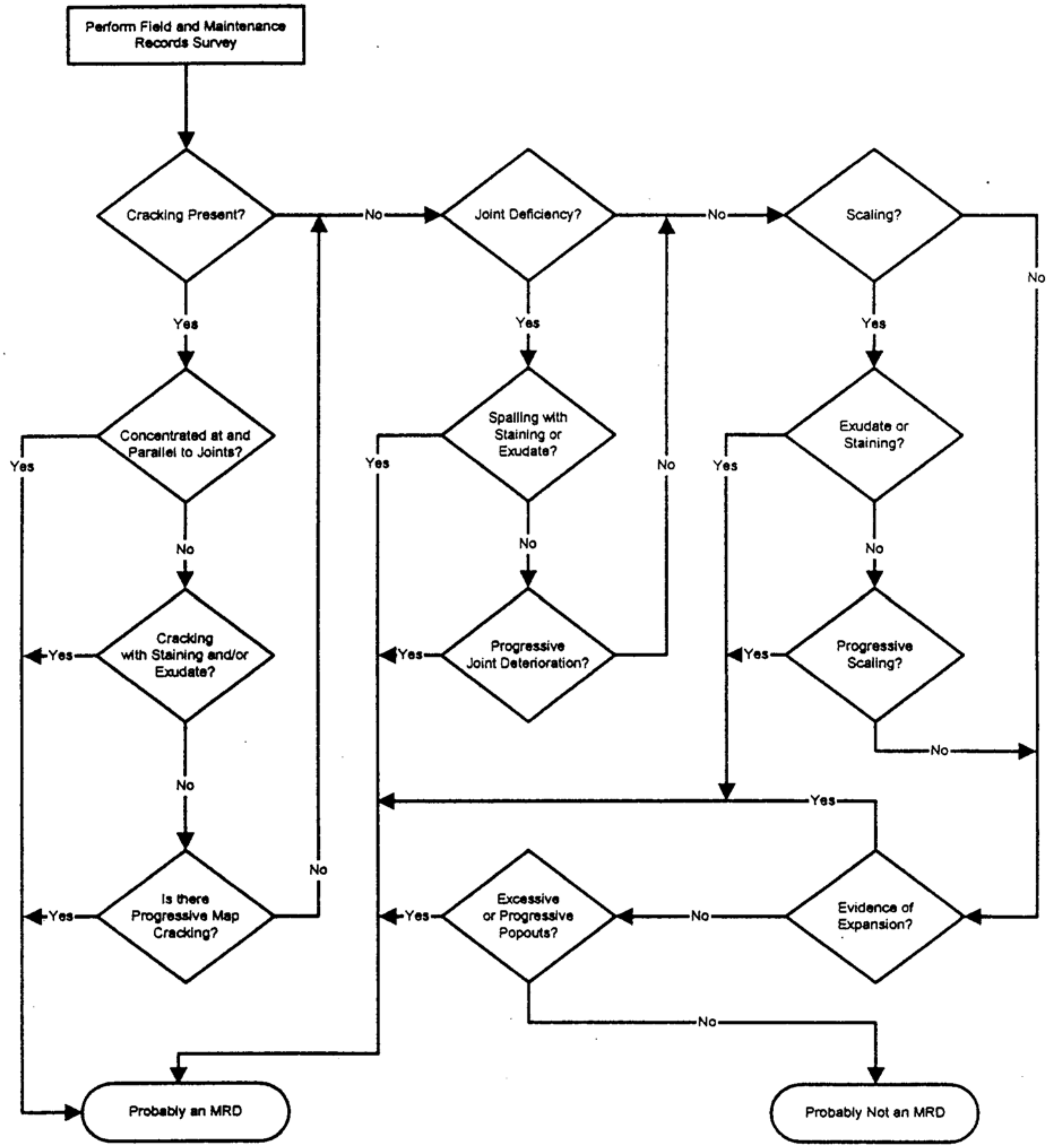


Figure 2. Flow Chart for Evaluating for Presence of MRD (from ref. *2*)

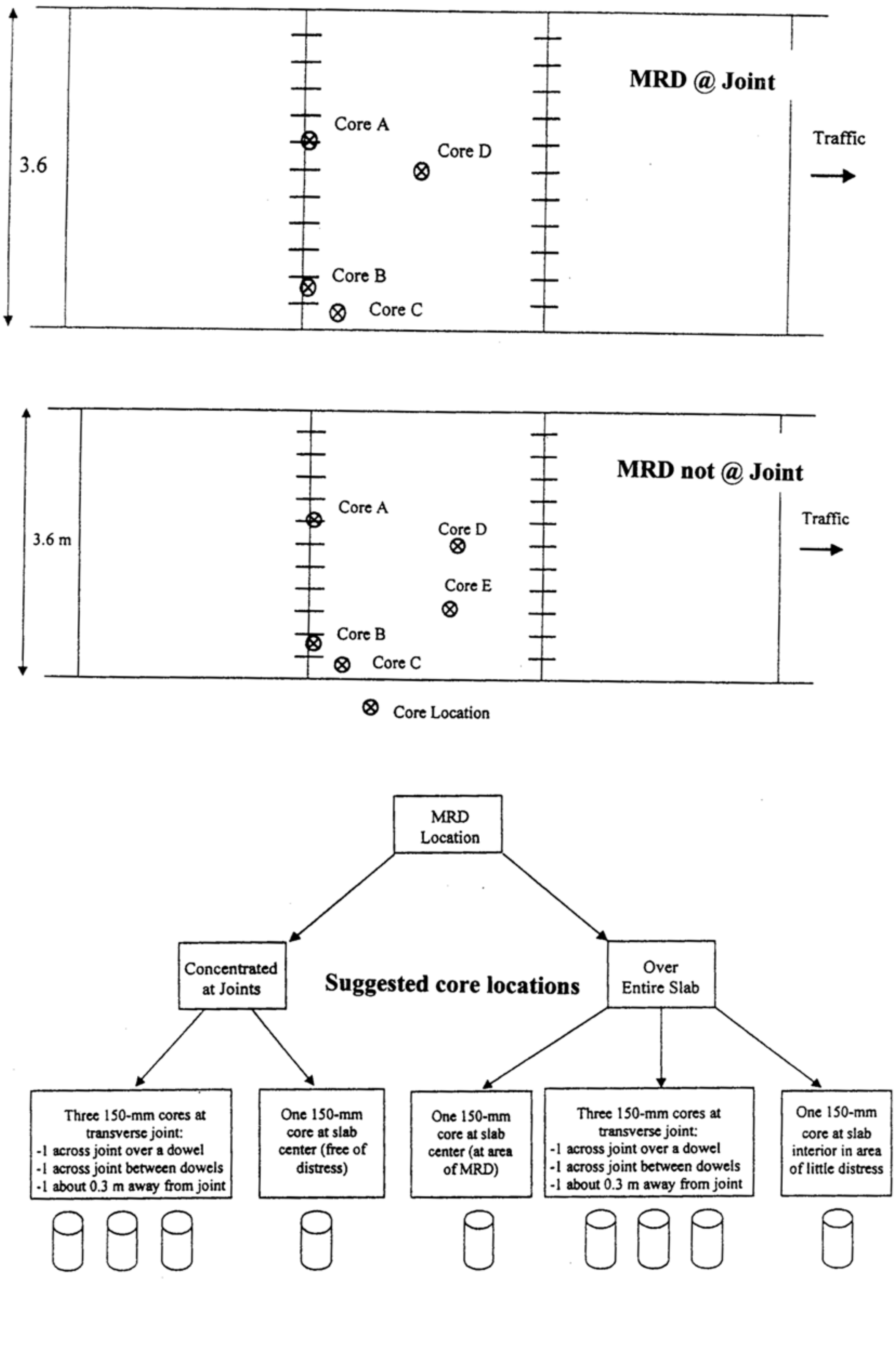


Figure 3. Details of Coring Requirements for Pavements Suspected of MRD (adapted from ref. *2*)

Current Iowa Department of Transportation Methods of Pavement Distress Evaluation

Pavement distress and deterioration of roadways is detected by several methods in the state of Iowa. On a small scale or in isolated cases, distress is determined through observation by Iowa DOT personnel familiar with the surface symptoms such as shadowing of transverse joints. Although pavement distress is typically the result of traffic and environment conditions, it may also occur due to construction or materials problems. This distress is usually first seen in low-lying wet areas with poor subbase drainage. Most commonly, the method of evaluation is visual examination with subsequent coring of the pavement. The Iowa DOT has a specialized-coring vehicle for collecting pavement cores. Once the cores are taken, sections of the cores are cut and polished to reveal macroscopic and microscopic deterioration of either the cement paste or the aggregate. Deterioration may be either in the form of physical deterioration of the aggregate, resulting from freeze-thaw failure due to clays or a bad pore system, or poor construction practices that produced early desiccation fractures (shrinkage cracks) in the surface of the pavement.

On a statewide basis, pavement condition is determined as part of the Pavement Management System (PMS), which was begun in the early 1980s. The PMS includes an online database of pavement condition and project history. The database includes inventory and history data in addition to pavement condition data. The database and associated analyses algorithms are used to assist in evaluating and maintaining pavements, to provide pavement and material histories, and to determine service histories for approvals of new aggregate sources. Pavement condition assessment test methods consist of structural integrity, ride quality, friction testing, and surface distress. Pavement management sections are listed by county, highway, and route direction. Current pavement assessment test methods include pavement distress surveys and other test data, summarized below, to formulate the relative index approach (RIA) value.

Pavement Distress Surveys

Between 1969 and 1999, manual network-level distress surveys were visually conducted to help assess the condition of the pavement network. Distress surveys were collected every two years, to quantify longitudinal and transverse cracking, patching, rut depths, and faulting. D-cracking was also evaluated by rating the closely spaced crescent shaped hairline cracks (distress cracking) from 1 to 5. A 1 is some evidence at joints but no maintenance required. A 5 indicates severe deterioration requiring regular maintenance and in some cases full-depth patching.

The manual distress survey has been replaced by an automated data analyses process developed by Roadware Group, Inc. Roadware uses a vehicle called the Roadware Automatic Road Analyzer (ARAN). This vehicle collects digital video images of the road surface using two back-mounted cameras and strobes to provide uniform lighting. The image analysis of the video is performed later at workstations using Roadware’s automated crack detection system called “WiseCrax.” The image analysis program detects crack length, width, orientation (longitudinal or transverse), and crack area. From this, the program classifies the cracks according to type, extent, and severity to generate summary statistics and crack maps. The number of patches and patch area is also determined. Some problems result because the Iowa DOT only receives summary data and does not receive the video images making verification of the data difficult.

Beginning in 2003 the Iowa DOT will be purchasing and analyzing a portion of the Roadware video images to evaluate the summary data.

Additionally, videologs of the highway surface and signage are produced annually and are principally used to correct milepost and signage problems, although the videologs can be used to check and confirm pavement distress. The videologs are produced using an Iowa DOT vehicle that was purchased from Mandli Communications, Inc.

Structural Testing Using the Road Rater or the Falling Weight Deflectometer

Structural testing of pavements is used to help determine rehabilitation strategies and pavement overlay designs or to determine the need for pavement replacement. To evaluate pavement structure, Iowa has used the "Road Rater" since 1976. The Road Rater operates by measuring the vertical displacement of a pavement surface in response to a cyclic force of 2500 pounds at a frequency of 30 cycles/second. The pavement response is detected by velocity transducers and recorded by on-board computer. Each measurement is a function of the pavement thickness, pavement resilient modulus or modulus of elasticity, and the subgrade resilient modulus at the test location. This information can be used to model the pavement response to vehicle wheel loading and be used to help predict pavement performance. The falling weight deflectometer (FWD) operates on similar principles, producing a load pulse that simulates the effect of a moving vehicle on the pavement surface by dropping a mass from a set height to generate a dynamic load. A load cell measures the applied load. The load is transmitted to the pavement through a plate, resulting in a deflection of the pavement surface. The deflection is measured by geophones, giving a picture of the deflection bowl that allows evaluation of pavement layers. With these data it is possible to perform structural analyses, estimate expected service life, and assist with maintenance or repair strategies.

Quality of Ride Testing using the High-Speed Laser Profiler

Pavement surfaces have been measured in Iowa using the high-speed laser profiler since 1990 to quantify the "quality of ride." The profiler measures the relative elevations of each wheel path using lasers. Accelerometers are used to remove the motion of the vehicle so that an elevation is obtained. An online computer stores elevation data while testing at highway speeds. The International Roughness Index (IRI) is then computed (shown in units of meters/kilometer), reviewed, and entered into the Pavement Management System.

Friction Testing

Friction testing is performed to measure the wet-weather friction properties of pavement surfaces. Iowa has been doing friction testing since 1970. The test apparatus uses a trailer with an independent wheel, which can be locked while driving at 40 mph (ASTM E 274). Water is sprayed on the road ahead of the test wheel. As the tire is locked, strain gauges determine the force required holding the tire locked. An on-board computer records the sensor data. This data is reprocessed on a desktop then entered into the PMS. The Friction Number is the force divided by the weight of the left wheel times 100. These normally range between 40 and 70 with higher numbers representing better friction values. Sections of the primary road system are tested on a variable schedule that depends on traffic load, classification, and previous friction values.

Summary for Field Data Collection Activities

In summary, it would be desirable to combine as many of the various components of the overall survey process into tasks that are easy to perform and document. For example, it would be advantageous to conduct preliminary surveys that are highly automated and do not require traffic control. It would be ideal if the preliminary surveys (i.e., the shoulder, the project layout, and the distress surveys as listed in Figure 1) could be conducted without leaving the office. This could be done by the proper use of pavement management information. However, it would also be better if we could look below the surface of the pavement to inspect for distress that is not yet apparent. That is the thrust of this research project, to provide pavement engineers with some additional tools that will streamline the identification and documentation of distress in PCC pavements. Once we have found the distress then we can refine the analysis to see what caused it.

OVERVIEW OF NONDESTRUCTIVE TESTING

There are many different definitions available for the term "nondestructive testing." The techniques are employed in a large number of different disciplines, and hence, each discipline has created their own specific definition of NDT. A broad definition of the term is as follows (*6*):

> Nondestructive testing (NDT), nondestructive evaluation (NDE), and nondestructive inspection (NDI) are the terms used in this connection to represent the techniques that are based on the application of physical principles employed for the purpose of determining the characteristics of materials or components or systems and for detecting and assessing the inhomogeneities and harmful defects without impairing the usefulness of such materials or components or systems.

This definition is interesting because it elucidates the fact that nondestructive tests can be directed at different levels of investigation. For example, consider a typical pavement section as shown in Figure 4. NDT methods can be utilized to measure properties or inspect for defects in the in-situ materials, the different components (such as the dowel bar placement in the slab), or the whole pavement system from top to bottom (pavement slab, base course, and subgrade). This robustness has made NDT an essential part of modern manufacturing processes. However, this is not meant to imply that NDT methods do not have their drawbacks or that they will quickly replace destructive testing. It is important to note that concrete testing has historically been based on destructive testing. A summary of the advantages and disadvantages of destructive and nondestructive testing is given in Table 1.

Summary of Existing Methods

Many methods exist for the nondestructive testing of concrete-based materials. Excellent summaries exist (*7, 8, 9, 10*), and for the purpose of this report the nomenclature of Carino (*10*) will be employed (see Figure 5). In fact, the interested reader can gain considerable insight into the rapid development in the field by a quick reading of the chapter pertaining to nondestructive testing in the ASTM special technical publication on concrete (*9, 11, 12, 13*). This series, which is published approximately each decade, clearly shows that NDT has evolved from a technique that was initially directed at strength estimation, into one that employs a wide variety of test methods that are directed at determining both material properties and assessment of the condition of in-situ concrete. The NDT directed at strength determination will not be discussed in this report. Interested readers should consult Malhotra (*9*) or Carino (*10*) for a detailed overview of the various techniques that are available. Most of these techniques are highly refined and many have been standardized. However, these techniques are of little interest to this study because they are directed at strength determination rather than the identification of materials-related distress (MRD is normally a durability issue rather than a strength issue); only the test methods that are directed at condition assessment (see the right half of Figure 5) will be considered further.

Table 2 summarizes the various methods that are currently listed in ACI 228.2R-98 (*7*). The 20 different tests can be sorted into subgroups that have a potential for identification and/or characterization of MRD. Also, several ground rules will be applied during the sorting process.

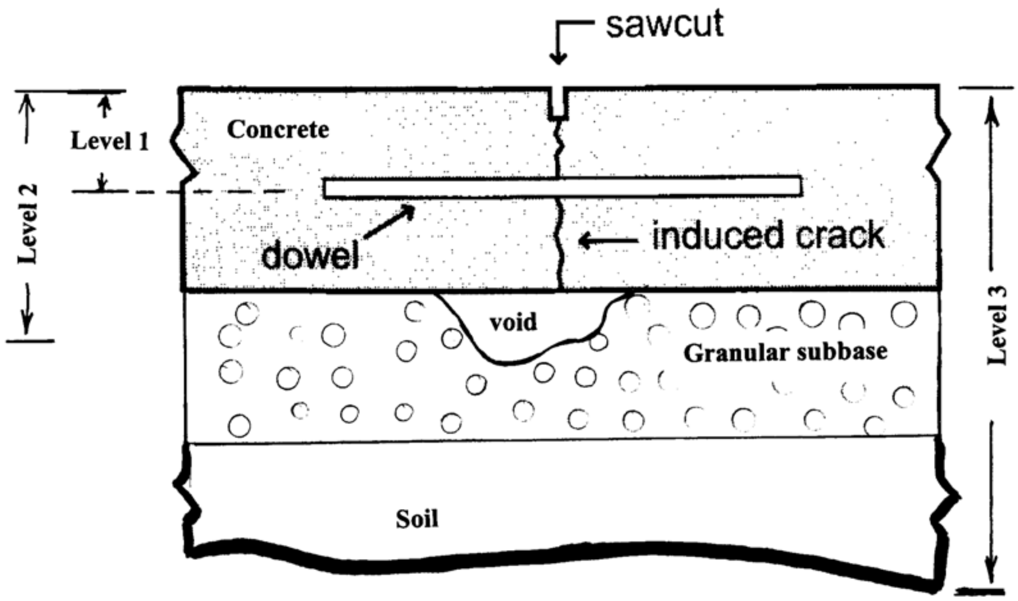


Figure 4. Illustration of Directing NDT at Different Levels of Investigation

Table 1. Comparison of Destructive and Nondestructive Tests (adapted from ref. *6*)

Destructive Tests	**Nondestructive Tests**
Advantages: • Measurements are direct and reliable. • Usually quantitative measurements. • Correlation between test measurements and material properties are direct.	Limitations: • Measurements are indirect; reliability is to be verified. • Usually qualitative measurements; measurements can also be done quantitatively. • Skilled judgment and experience are required to interpret indications.
Disadvantages: • Tests are not made on the objects directly; hence, correlation between the sample specimen used and object needs to be proved. • A single test may measure only one or a few of the properties. • In-service testing is not possible. • Measurement of properties over a cumulative period of time cannot readily be possible. • Preparation of the test specimen is costly. • Time requirements are generally high.	Advantages: • Tests are made directly on the object; 100% testing on actual components is possible. • Many nondestructive test methods can be applied on the same part, and hence many or all properties of interest can be measured. • In-service testing is not possible. • Repeated checks over a period of time are possible. • Very little preparation is sufficient. • Most test methods are rapid.

In-situ detection suggests the use of nondestructive testing or remote imaging.

Estimate mechanical properties (typically strength) ⇐ **Classical NDT** ⇒ **Condition assessment (find hidden flaws)**

Typical tests include

- pullout test
- break-off test
- pin penetration
- pull-off test
- tork test
- maturity

Typical tests include

- visual inspection
- infrared thermography
- ground penetrating radar
- stress wave propagation
- nuclear methods
- magnetic and electrical
- penetrability methods

Figure 5. Common Methods Available for Nondestructive Testing (adapted from ref. *10*)

Table 2. Summary of Nondestructive Test Methods (adapted from ref. *7*)

Section	Method and Principle	Applications
2.1	**Visual inspection**—Observe, classify, and document the appearance of distress on exposed surfaces of the structure.	Map patterns of distress such as cracking, spalling, scaling, erosion, or construction defects
2.2.1	**Ultrasonic pulse velocity**—Measure the travel time of a pulse of ultrasonic waves over a known path length.	Determine the relative *condition* of concrete based on measured pulse velocity.
2.2.2	**Ultrasonic-echo**—Transducer emits short pulse of ultrasonic waves, which is reflected by opposite side of member or an internal defect; arrival of reflected pulse is recorded by an adjacent receiver, and round-trip travel time is determined.	Locate delaminations and voids in relatively thin elements. Primarily a research tool.
2.2.3	**Impact-echo**—Receiver adjacent to impact point monitors arrival of stress waves as they undergo multiple reflections between surface and opposite side of plate-like member or from internal defects; frequency analysis permits determination of distance to reflector if wave speed is known.	Locate a variety of defects within concrete elements such as delaminations, voids, or honeycombing, or measure element thickness.
2.2.4	**Spectral analysis of surface waves**—Impact is used to generate a surface wave and two receivers monitor the surface motion; signal analysis allows determination of wave speed as a function of wavelength; inversion process determines elastic constants of layers.	Determine the stiffness profile of a pavement system. Also used to determine depth of deteriorated concrete.
2.3.1	**Sonic-echo**—Hammer impact on surface and a receiver monitors reflected stress wave; time-domain analysis used to determine travel time.	Determine the length of deep foundations (piles and piers); determine the location of cracks or constrictions (neck-in).
2.3.2	**Impulse-response**—Text is similar to sonic-echo method except that signal processing involves frequency-domain analysis of the received signal and the impact force history.	Determine the length of deep foundations (piles and piers), location of cracks or constrictions (neck in). Provides information on the low-strain dynamic stiffness of the shaft/soil system.
2.3.3	**Impedance logging**—Test is similar to sonic-echo or impulse-response, but the use of more complex signal analyses (time and frequency domains) allows reconstructing the approximate shape of the deep foundation.	Determine the approximate two-dimensional shape of the deep foundation.
2.3.4	**Cross-hole sonic logging**—Analogous to the ultrasonic pulse velocity test, but transducers are positioned within tubes cast into the deep foundation or holes drilled after construction.	Determine the location of low-quality concrete along the length of the shaft and between transducers. With drilled holes, permits direct determination of shaft length.
2.3.5	**Parallel seismic**—Receiver is placed in hole adjacent to the foundation; foundation is struck with a hammer and signal from receiver is recorded; test is repeated with receiver at increasing depth.	Determine the foundation depth and determine whether it is of uniform quality.

Table 2. Summary of Nondestructive Test Methods (adapted from ref. *7*), continued

Section	Method and Principle	Applications
2.4.2	**Direct transmission radiometry**—Measure the intensity of high-energy electromagnetic radiation after passing through concrete.	Determine in-place density of fresh or hardened concrete. Locate reinforcing steel or voids.
2.4.3	**Backscatter radiometry**—Measure the intensity of high-energy electromagnetic radiation that is backscattered (reflected) by the near surface region of a concrete member.	Determine in-place density of fresh or hardened concrete.
2.4.4	**Radiography**—The intensity of high-energy electromagnetic radiation that passes through a member is recorded on photographic film.	Locate reinforcing and prestressing steel, conduits, pipes, voids, and honeycombing.
2.4.5	**Gamma-gamma logging**—See direct transmission and backscatter radiometry.	Locate regions of low density along length of foundation.
2.5.1	**Covermeter**—A low frequency alternating magnetic field is applied on the surface of the structure; the presence of embedded reinforcement alters this field, and measurement of this change provides information on the reinforcement.	Locate embedded steel reinforcement, measure depth of cover, and estimate diameter of reinforcement.
2.5.2	**Half-cell potential**—Measure the potential difference (voltage) between the steel reinforcement and a standard reference electrode; the measured voltage provides an indication of the likelihood that corrosion is occurring in the reinforcement.	Identify region or regions in a reinforced concrete structure where there is a high probability that corrosion is occurring at the time of the measurement.
2.5.3	**Polarization methods**—Measure the current required to change by a fixed amount the potential difference between the reinforcement and a standard reference electrode; the measured current and voltage allow determination of the polarization resistance, which is related to the rate of corrosion.	Determine the instantaneous corrosion rate of the reinforcement located below the test point.
2.6	**Penetrability methods**—Measure the flow of a fluid (air or water) into concrete under prescribed test conditions; the flow rate depends on the penetrability characteristics of the concrete.	Compare alternative concrete mixtures. Primarily research tools, but have the potential to be used for assessing adequacy of curing process.
2.7	**Infrared thermography**—The presence of flaws within the concrete affects the heat conduction properties of the concrete, and the presence of defects are indicated by differences in surface temperatures when the test object is exposed to correct ambient conditions.	Locate delaminations in pavements and bridge decks. Also widely used for detecting moist insulation in buildings.
2.8	**Radar**—Analogous to the ultrasonic-echo methods except that electromagnetic waves are used instead of stress waves; interface between materials with different dielectric properties results in reflection of a portion of incident electromagnetic pulse.	Locate metal embedments, voids between pavements, and regions of high moisture contents; determine thickness of members.

Pavements are a rather special case of concrete construction because they have such an immense size, and hence, they require special considerations (i.e., NDT methods that are perfectly suited to bridges may not be adequate for pavements). The ground rules can be summarized as follows:

- should be able to inspect large areas efficiently (i.e., a “global” inspection method)
- should have a minimal impact on traffic flow
- should be relatively easy to interpret

Application of the ground rules and other practical considerations to the NDT methods summarized in Table 2 yielded the subgroup listed in Table 3. Each of the techniques is briefly described below.

Table 3. Nondestructive Test Methods Applicable to This Study (adapted from ref. *7*)

Test Method—Use	Strengths	Weaknesses
Visual inspection—Identification and categorization of distress.	Sensitive to fine details.	Subjective—often difficult to quantify.
Impact-echo—Locates delaminations and voids and measures element thickness.	Access to only one face is required; equipment is commercially available; capable of locating a variety of defects; does not require coupling materials.	Experienced operator is required; current instrumentation limited to testing members less than 2 meters thick.
Spectral analysis of surface waves—Determines the stiffness profile of a pavement system and determines depth of deteriorated concrete.	Capable of determining the elastic properties of layered systems, such as pavements, interlayered good and poor-quality concrete.	Experienced operator is required; involves complex signal processing.
Infrared thermography—Locates delaminations or voids.	A global technique applicable to large surveys; results provide an indication of the percentage of deteriorated area in the survey region.	Equipment is expensive; test response varies with environmental conditions so testing may be restricted; cannot measure the depth or thickness of a subsurface anomaly; experienced operator is required.
Ground penetrating radar—Locates voids, metal objects, and areas of high moisture content.	A global technique applicable to both large and small surveys; sensitive to the presence of embedded metal objects; able to penetrate across concrete-air interfaces; sensitive to the presence of moisture.	Region irradiated by the antenna is limited to cone-shaped volume directly below antenna; cracks not easy to detect unless moisture is present; experienced operator is required to operate equipment and interpret results.

Visual Inspection

As was alluded to earlier in this report, the human eye is one of the most sensitive detection methods that are available for locating materials-related distress. However, the human eye is also prone to subjective rather than objective measurements. Considerable training and experience are needed to overcome this lack of objectivity. In addition, the rate of data collection is slow and humans lack the ability to adequately archive the observations. Hence, many departments of transportation have adopted video or image-based data collection strategies. This change has been linked to the economy promised by new technology (*14*) and the budget constraints placed on state departments with regard to staff reduction or hiring restrictions (*15*). Humans may eventually be totally replaced for data collection, but the human eye will remain a substantial part of data evaluation and interpretation for a good time to come.

Stress Wave Propagation Techniques

Both the impact-echo technique and the spectral analysis of surface waves (SASW) technique are commonly classified as stress wave propagation techniques. They are similar because a stress wave is induced via mechanical impact on the object of interest, and surface displacement is measured (impact-echo) or surface motion at two points a known distance away from the source is measured (SASW). The two techniques utilize information obtained from different types of waves, and hence, they require different equipment and different signal processing strategies. The following summaries of the two methods are very simplistic and interested readers should consult other sources for more detailed information (*10, 16*).

Impact-Echo

The basic principle of the impact-echo technique is illustrated in Figure 6. The solid object is struck sharply with a mechanical impactor (note that impact duration is an important variable in this technique). The stress pulse generates a surface wave (R-wave) and it also travels through the solid as elastic waves (P- and S-waves). When the stress waves reach a flaw or a material boundary in the solid, the waves are reflected and refracted. The reflected waves can be detected at the surface of the solid by an appropriate receiving transducer that measures a displacement. The time-displacement information can be viewed and saved for later inspection with appropriate hardware. Typically an oscilloscope or waveform analyzers are used for this purpose. The waveforms are normally converted to the frequency domain prior to interpretation. ASTM C 1383 (*17*) summarizes the apparatus required, general procedure, and a description of potential errors that are common to the test when it is used to measure the thickness of concrete members. It is important to note that acoustic coupling of the transducer to the solid is required in this technique; however, the method has been refined to the point that liquid couplants are not required. Hence, the technique can be used to investigate relatively large areas. Sansalone and Carino (*16*) describe a case study in which the impact-echo technique was used to detect defects (voids) in a 6-inch thick concrete slab that had been constructed for an ice-skating rink.

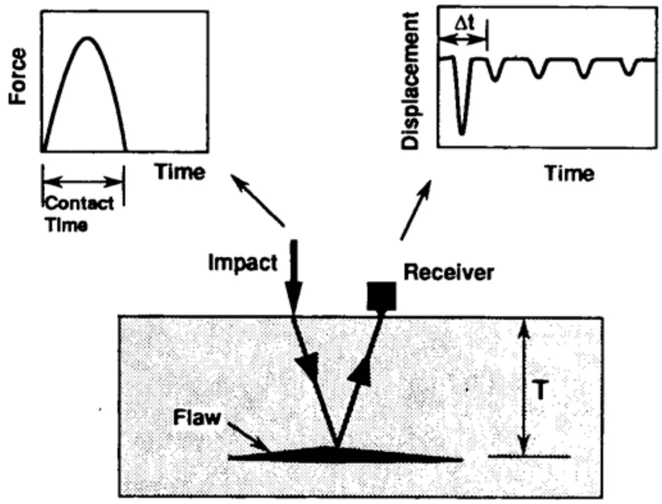


Figure 6. Illustration Depicting the Impact-Echo Method (from ref. *17*)

Spectral Analysis of Surface Waves

The principle behind the SASW technique is illustrated in Figure 7. Typically, a hammer is used to strike the surface of the solid object and two receiver transducers are used to detect the surface wave (R-wave) generated by the impact. The receiver output is stored in a two-channel spectral analyzer. The receiver transducers typically have to be varied over several different spacings to provide reliable test results. The output from the test is complex and is generally subjected to a considerable amount of signal processing. Interpretation of the information requires a high level of expertise, and this has constrained the use of the technique. However, the technique has been successfully used to evaluate pavements. Nazarian, Yuan, and Baker (*18*) describe the use of a seismic pavement analyzer based on the SASW technique. The device was used to conduct quality control and quality assurance studies on a PCC roadway. The device was contained on a trailer that could be towed from spot to spot; this provided the researchers with good mobility so that large areas could be probed. However, since the receiver transducers must contact the surface of the pavement during the measuring cycle, the device still interrupted the flow of traffic at the site. The article also mentioned a smaller more portable version of the seismic pavement analyzer.

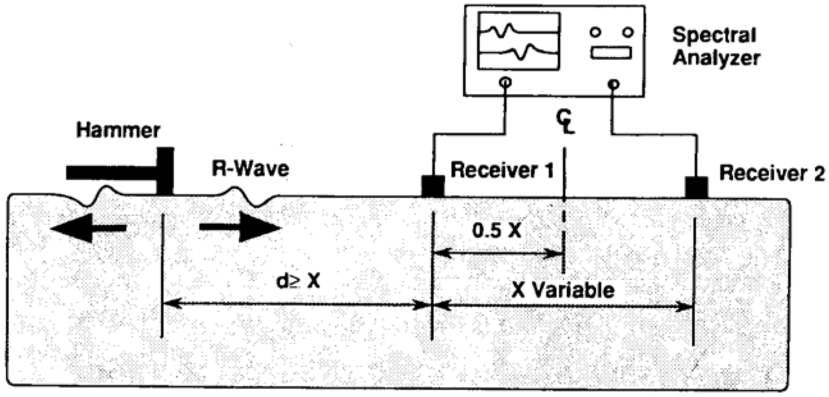


Figure 7. Illustration of Spectral Analysis of Surface Waves Technique (from ref. *17*)

Infrared Thermography

Infrared thermography is a technique that is used to measure the temperature of the surface of an object. The method relies on the fact that the heat flow into (or from) and object is sensitive to the presence of voids (or other subsurface features of low thermal conductivity) in the solid. This concept is illustrated in Figure 8 (*19*). Hence, when a very sensitive infrared scanner is used to measure the temperature of the surface of the solid, hot spots and cold spots (temperature differences) appear in the thermogram. This leads one to infer that a flaw is present. The method is commonly used to evaluate concrete bridge decks for delaminations and ASTM D 4788 (*20*) standardizes the procedure for such surveys. Weil (*21*) indicates that the method has been applied to a broader range of applications (e.g., highways, dams, and airport taxiways to name a few) and presents case studies for several applications. The method is applicable to testing large areas of concrete (i.e., a global inspection technique), and recent rapid improvements in computer technology have made data collection, analysis, and interpretation much simpler and more reliable. Proper use of the technique can reveal the presence, location, and horizontal extent of a void; however, the method is limited by its inability to determine the depth to the void or vertical extent of the void.

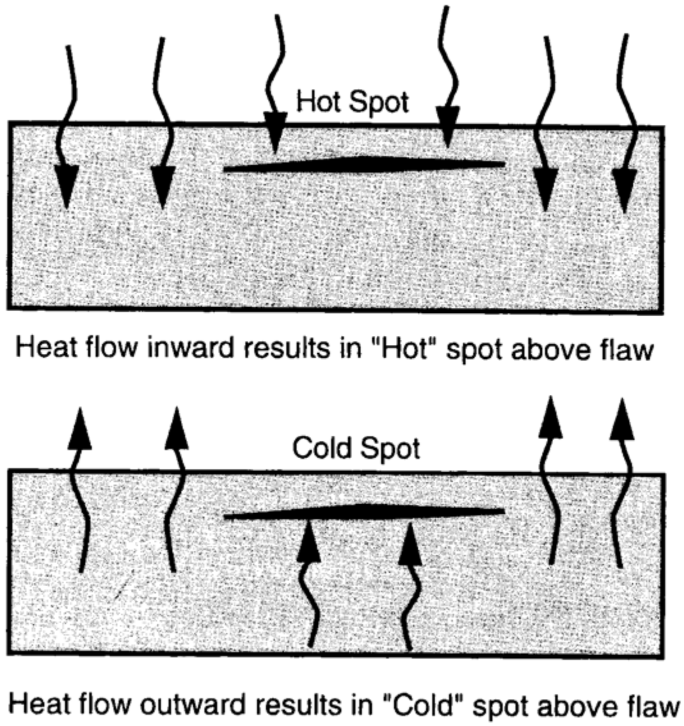


Figure 8. Illustration of the Principle behind Infrared Thermography (from ref. *18*)

Ground Penetrating Radar

Ground penetrating radar (GPR) is commonly referred to by several different terms. European literature often uses the terms “impulse radar” or “short pulse” while early literature often just used the term “radar” or “penetrating radar.” In all instances the technique is the same. A beam of radio waves is directed at a specimen and a receiver collects the waves that are reflected back (*22*). The basic principle, which is illustrated in Figure 9, is very similar to the stress wave propagation techniques. However, in the GPR method, waves of electromagnetic radiation are substituted for stress waves. Hence, the instrumentation required to conduct a GPR survey is roughly similar to that which would be used to conduct an impact-echo survey. Such equipment would include a transmitter, a receiver, and digital signal processing and recording equipment. The receiver-transmitter assembly is commonly referred to as an antenna, and it can be ground coupled or air-launched. Complete systems are available for rent or purchase and can be custom configured to meet the needs of most users. Systems configured with an air-launched antenna have been used to collect detailed GPR data at highway (about 50 to 60 mph) speeds.

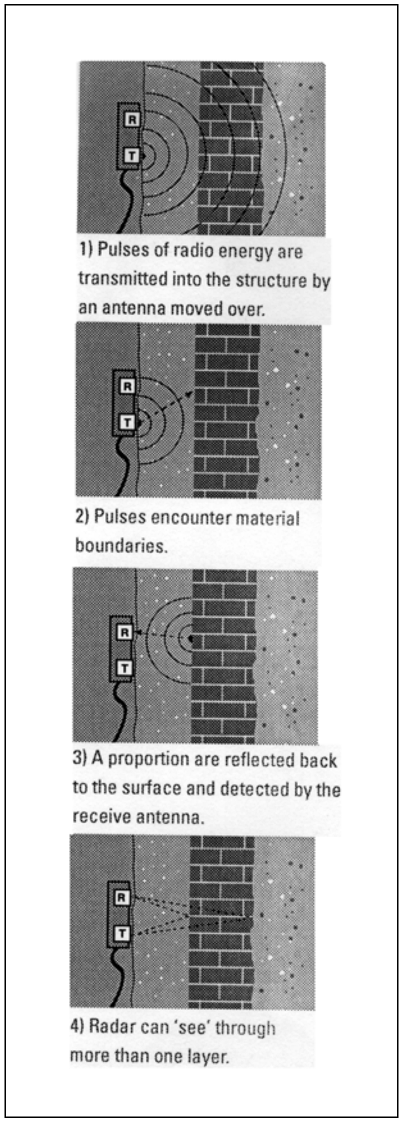


Figure 9. Diagram Explaining the Principal behind Ground Penetrating Radar (from ref. *22*)

Handbooks give excellent summaries of the basics of GPR theory and potential applications of the technique (e.g., see references *7, 10, 19,* and *23*), so this report will not dwell on these details. In addition, a standard test method is available (ASTM 4748) for the measurement of pavement layer thickness (*24*), so it is evident that this particular field is rapidly maturing. Initially, GPR was considered as a technique for the evaluation of bridge decks and much work is still being conducted in that area (*25, 26*). However, investigators in several states—Florida, Louisiana, Missouri, and Texas to name only a few—have conducted network-length studies on pavements (*27, 28, 29, 30, 31*). Hence, the thrust has been to push the technology into the broader areas of pavement management or preventative maintenance (*32, 33, 34*). On a project-level scale, investigators have focused at detecting smaller and smaller details in the pavement structure. For example, researchers describe a study pertaining to the detection of voids related to workability problems (resulting in segregation and/or poor consolidation) in a concrete pavement (*35*).

One constraint in the use of GPR technology is that the Federal Communications Commission has ruled to authorize the deployment of ultra-wideband technology (*36*). The ruling was deemed necessary to avoid inference with radio signals from commercial stations. To date, the ruling has only resulted in the registration of GPR devices; however, the commission is still evaluating solutions to the dilemma. An overview of the ruling and recent amendments pertinent to the GPR industry are available on the internet (*37*).

Gaps or Weaknesses in Nondestructive Evaluation Techniques

The weaknesses in the various nondestructive techniques of interest to this study have for the most part been summarized in Table 3. The major weaknesses in most of the methods have little to do with the apparatus. Equipment for most of the techniques has gone through rapid improvement due to miniaturization or improvements related to the rapid rise in speed and storage components for personal computers. And it appears that the equipment will continue to improve. Rather, the techniques all tend to be weak in the area of interpretation. At present, interpretation is limited to the realm of people who have considerable expertise in the field of NDT. This is generally good; however, it is also similar to the state of data interpretation in the field of X-ray diffraction during the 1960s, 1970s, and 1980s. During that time, few people other than experts dealt with the identification of crystalline solids via diffraction techniques. However, as the personal computer evolved so did the software that was commonly used to help identify the materials. The result is that today technicians are able to identify most common compounds via X-ray diffraction (actually most computers are able to do this with little user help!). The point this author is trying to make is that hardware improvements did not drive the transition. Instead, software improvements greatly reduced the amount of expertise needed to master the task. This also appears to be what is constraining the use of NDT methods. In addition, software developers should strive to provide for visualization of data rather than simple interpretation. Visualization in this instance refers to the capability to provide engineers with graphical information in a format that simplifies the diagnosis of the problem (typically three-dimensional images). Data fusion methods (*38*) may also help to improve the reliability of the NDT techniques that are commonly applied in this field of research.

Emerging Technologies

Phase-measurement laser radar (Ladar) technology has been developed by Phoenix Scientific, Inc. (*39*). The device has very promising technical capabilities (very precise distance measurements at extremely high sampling rates), and it has been incorporated into the pavement profile scanner (PPS) that has been developed by Mandli Communications, Inc. (*40*). The PPS system has the potential to collect transverse (rutting), longitudinal (faulting), and distress profiles (cracking) while traveling at highway speeds. The technology is very new and the hardware and software are still actively developed. However, the device is functional and available for testing.

Summary and Selection of Field Techniques for "Proof of Concept" Testing

In summary, an overview of NDT methods has been conducted with special emphasis directed at enhancing a pavement engineers' ability to detect and diagnose materials-related distress. Ground rules were applied to filter the many available techniques into a smaller subset that could be described in greater detail. The purpose of this section is to describe the methods that will be used in the preliminary round of "proof of concept" testing.

Previous research in the area of pavement distress suggests that a couple pieces of information are highly desirable for initiating any given survey. First, surface information pertaining to cracking or discoloration of the pavement surface is needed. Second, subsurface information about the integrity of the pavement slab and base material(s) is handy for ruling out structural-related cracking. For the purpose of this research project we will employ photolog images and the pavement profile scanner to provide information about cracks and displacements of the surface of the pavement slab. These two techniques have the ability to provide both qualitative and quantitative information about pavement distress. However, neither has the ability to differentiate cracking due to materials-related distress from structural cracking. Hence, another technique must be used to determine if flaws are present below the surface of the concrete slab. Stress-wave propagation techniques, infrared thermography, and ground penetrating radar are all capable of performing this function. The stress-wave propagation techniques appear to be particularly well suited to this type of study. Unfortunately, they lack the mobility needed to perform surveys on pavements while they are in service. Likewise, the infrared thermography technique is limited by its inability to detect defects near the base of the pavement slab. Only the ground penetrating radar technique currently has the ability to perform global surveys at reasonable speeds. Hence, it was chosen for further experimentation.

EQUIPMENT AND PROCEDURES

Initially, it was anticipated that the field phase of this study would consist of using two different technologies in a preliminary round of “proof of concept” testing. However, technical difficulties did not allow for the evaluation of the pavement profile scanner. Hence, the “proof of concept” testing for Phase 1 consisted only of ground penetrating radar. The GPR work was conducted using two different systems; this was done to evaluate hardware and software differences that were germane to the selection of such equipment during Phase 2 of the project.

GPR – Study 1

The preliminary ground penetrating radar study was conducted by Iowa State University (ISU) personnel. The study employed a Sensors & Software pulseEKKO 1000A system that was controlled by a laptop computer. The computer was connected to the hardware using a pE 1000 High Speed kit. Two different ground-coupled antennas (center frequencies of 900 MHz and 1200 MHz) were used during the study. The system was towed behind a slow moving vehicle, and hardware data collection constraints limited the speed of the vehicle to about 5 mph. The antenna was automatically triggered using a wheel odometer. The base setup is illustrated in Figure 10. The data collected during the fieldwork was evaluated using the comprehensive software package that was provided by the vendor (WIN EKKO Pro PC, EKKO 3D, EKKO MAPPER, and EKKO POINTER).

GPR – Study 2

The second ground penetrating radar study was conducted by Mike Scott of MGPS, Inc. The study employed a Geophysical Survey Systems, Inc. (GSSI) SIR-10 A+ ground penetrating radar system. Three different ground-coupled antennas (center frequencies of 400 MHz, 900 MHz, and 1500 MHz) were used for the study. The system was capable of collecting high-resolution data at highway speeds (55 to 60 mph). The base setup that was employed is illustrated in Figure 11.

Pavement Profile Scanner Study

Mandli Communications, Inc., was contacted to perform an extended field demonstration of their Pavement Profile Scanner. The systems were to be integrated into the existing Iowa DOT photolog van. The van was upgraded with the new device and delivered to the Iowa DOT in early November 2002. However, the firmware of the PPS system did not perform up to specifications and the field demonstration had to be canceled until the firmware problems could be fixed. This delay, coupled with the onset of winter weather, caused the study to be transferred into Phase 2 of the research project.

Figure 10. GPR Equipment Used for the ISU Surveys

Figure 11. GPR Equipment Used for the MGPS, Inc., Surveys

Field Sites

Four sites were selected for the “proof of concept” testing. The sites were chosen based on prior information about the sites. Site details are summarized below (see Table 4). Two of the sites exhibited aggregate-related distress, one site exhibited paste-related distress, and the final site exhibited no distress (control section). The site that exhibited paste-related distress was P 73, a stub from a section of US 20 (previously called Highway 520) that was constructed in 1987. That particular section of US 20 was covered with an asphalt overlay in 1996 because of “premature distress.” The stub exhibited severe map cracking that was similar to that observed on US 20.

Table 4. Field Sites Evaluated Using GPR Equipment

Site	Location	DOT Designation	Pavement Details	Distress Type
1	US 63, Poweshiek County	FN-63-4(6)-21-79	Constructed 1980	Aggregate related
2	Hwy 146, Poweshiek County	FN-146-2(6)-20-79	Constructed 1983	Aggregate related
3	P 73, Webster County	County road	Stub of Hwy 520	Premature distress
4	Hwy 175, Hamilton County	F-175-7(13)-20-40	Constructed 1980	None observed

RESULTS AND DISCUSSION

US 63, Poweshiek County

US 63 is a two-lane portland cement concrete pavement that was constructed with a design depth of 8.5 inches. Coarse aggregate from the Malcom mine was used in the pavement. This particular coarse aggregate is a crushed limestone that has a Class 2 durability rating, and hence, should show some distress after about 20 years of service. The GPR site was located about eight miles south of Tama, Iowa, near milepost marker 109.5. This section of pavement, shown partially in Figure 12, was surveyed using both sets of GPR equipment. Only the southbound lane of the pavement was evaluated in this study. Figure 12 also illustrates the paint markings where cores were to be extracted for pavement thickness and petrographic examination. Due to safety issues, the cores located near the center line of the pavement (i.e., near the intersection of the longitudinal and transverse joints) were not extracted from that region. Instead, they were taken about one-foot in from the shoulder of the pavement slab.

As anticipated, the pavement exhibited signs of distress such as cracking and staining at the joints. No cracking was noticed in other regions of the pavement slabs. Typical surficial pavement defects are shown Figure 13. Cores extracted from the pavement exhibited significant cracking (see Figure 14). Cores extracted from the pavement joints showed more deterioration than cores extracted from the mid-panel region. Petrographic examination of the cores indicated that the cracking was related to a failure of the coarse aggregate particles (see Appendix A).

The GPR survey conducted by MGPS, Inc., was about a half-mile in length. The data were collected while moving at about 50 mph. Several interesting response characteristics were noted in the GPR data that were collected from US 63 southbound. The most interesting was the phase change that alternates with a varying response magnitude at the expected location of the concrete-subgrade interface (see Figures 15 and 16). A red hash mark indicates the time that corresponds to the expected depth of the concrete-subgrade interface in the Figure 15 response. The red oval that is located at the corresponding depth highlights a response to the boundary between the concrete and subgrade that alternates phase very frequently in the direction of the data collection. This appears to be indicative of moisture gradients and/or variable support beneath the pavement. These anomaly responses are likely to be a result of either deteriorated concrete or poor support at the subgrade-concrete interface. Similar, less pronounced features are found along the length of the data presented in Figure 15. Figure 16 simply enlarges the scale of the horizontal axis and gives a better view of the region where distress was expected.

The GPR survey conducted by ISU personnel produced information similar to that obtained from the MGPS, Inc. survey. However, the ISU survey had a length of about 100 feet (30 meters) and was collected at a maximum speed of 5 mph. The main feature noted was the periodic signal at the boundary between the subgrade and the concrete slab (see Figure 17). Previous research has indicated that this GPR signal has been observed in pavements that exhibited d-cracking at the pavement joints (*41*). More work is needed to see if the extent of the distress can be accurately mapped using the GPR results.

Figure 12. Overview of US 63 Pavement Site (looking north near the core locations)

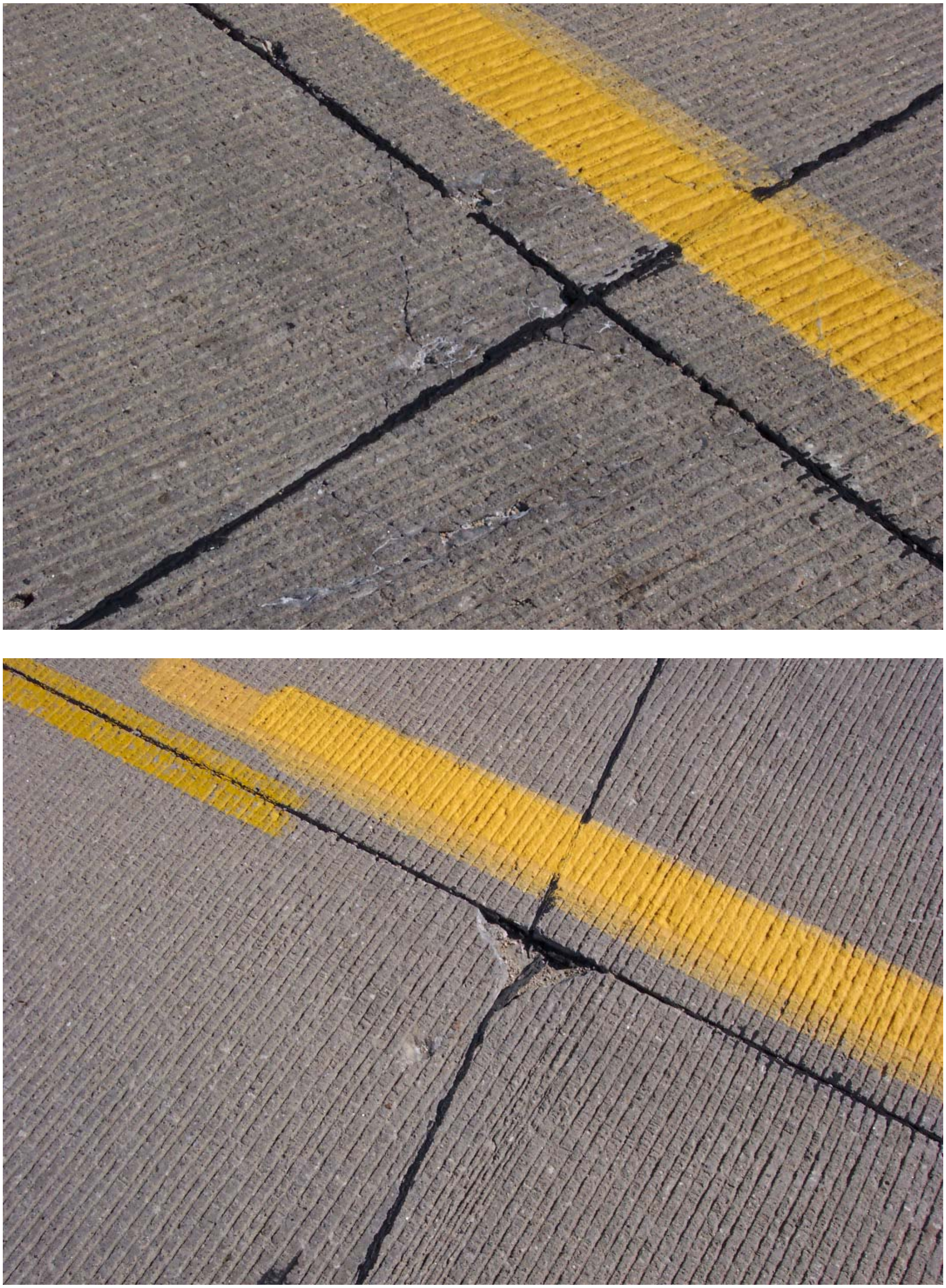


Figure 13. Typical Distress Features Noted on US 63 (note cracking in top photo)

Figure 14. Cores Extracted from US 63 (note deterioration at the base of the cores)

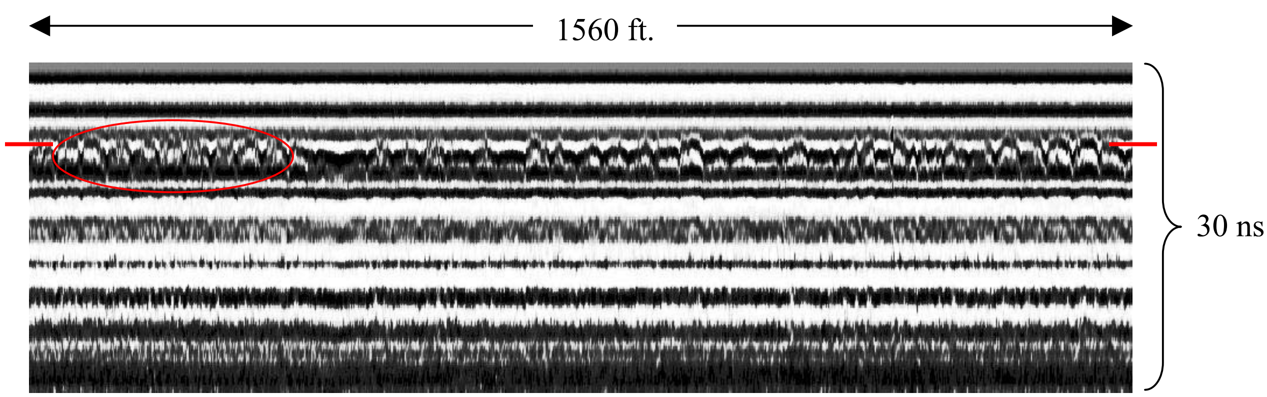


Figure 15. GPR Data from US 63 Southbound (900 MHz antenna)

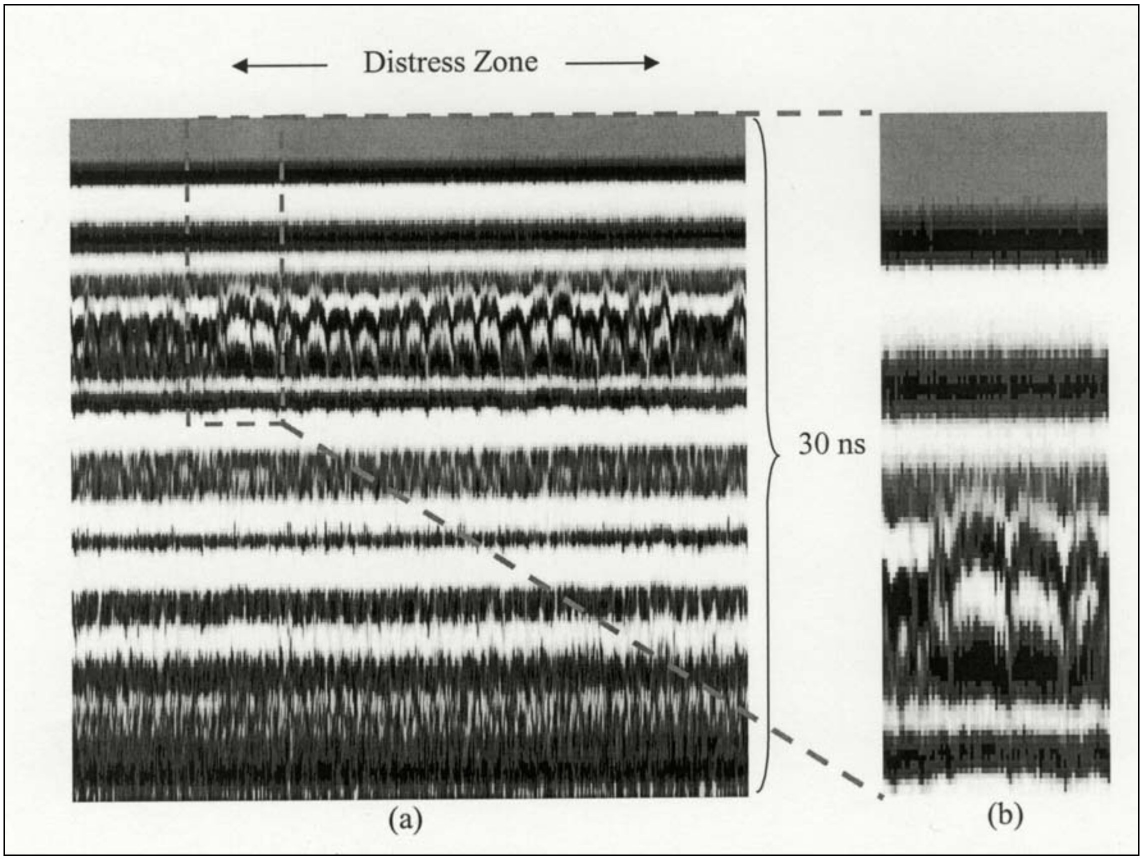
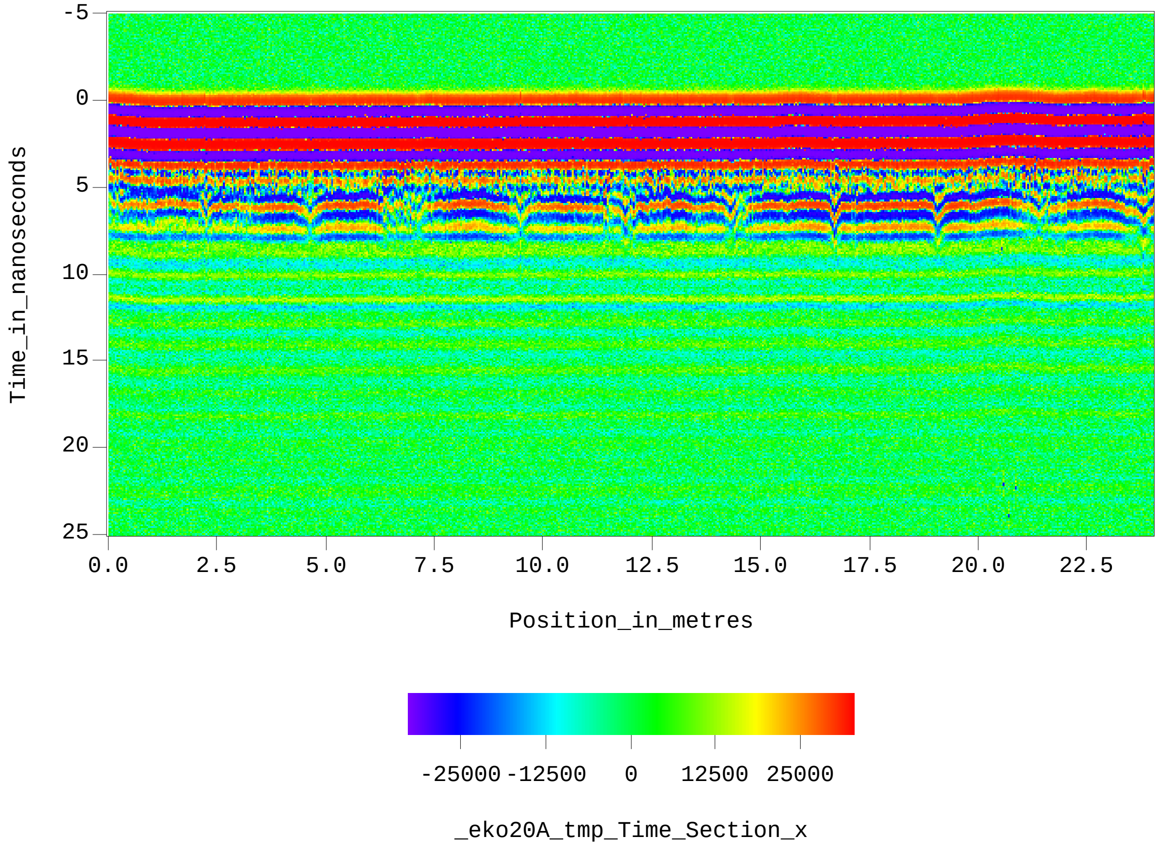


Figure 16. Magnified View of the GPR Data in the Distress Region (900 MHz antenna)

1200 MHz Antenna



900 MHz Antenna

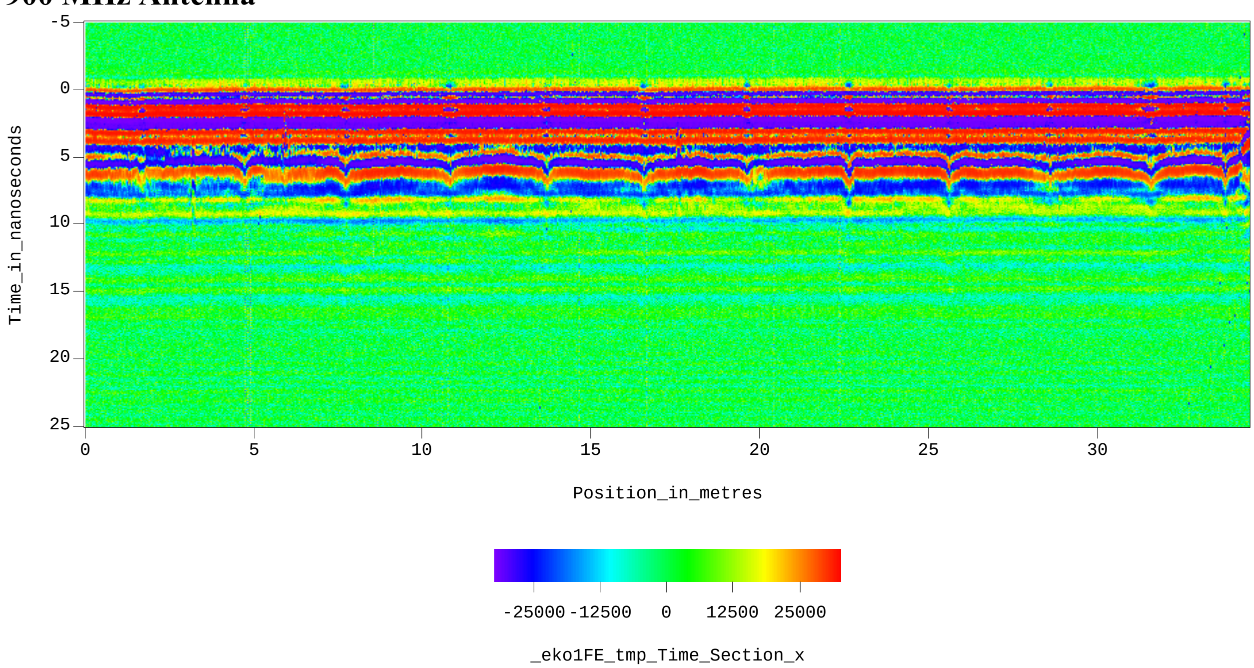


Figure 17. High-Resolution GPR Illustrating the Periodic Signal from the Base of US 63

Hwy 146, Poweshiek County

Highway 146 is a four-lane PCC pavement that was constructed with a design depth of 9.0 inches. Again, coarse aggregate from Malcom mine was used in the pavement. The GPR site was located near milepost marker 20 (see Figure 18). This is just north of exit 182 from I-80 so the road carries a good deal of traffic into the town of Grinnell. The pavement was surveyed using both sets of GPR equipment; however, only the data from the MGPS, Inc., survey will be presented because the data collection speed and test results were similar in both surveys (traffic limited the speed of the survey conducted by MGPS, Inc.). Only the northbound driving lane of the pavement was evaluated in this study. Five cores were extracted from the Highway 146 pavement slab. Four of the cores were taken from the joint area and one core was taken from the mid-panel area of the slab. Again, as noted above, the cores were extracted from near the shoulder of the pavement rather than near the center line of the pavement.

The pavement exhibited some signs of distress (note the patches in Figure 18); however, this was commonly limited to small spalls or corner breaks (see Figure 19). Little cracking was noticed in other regions of the pavement slabs. Cores extracted from the pavement exhibited cracking near the base of the slab (see Figure 20). Steel bars (3/8" nominal diameter) were observed in the bottom third of the cores extracted from the joint region of the pavement slab. Cores extracted from the pavement joints tended to show more deterioration than cores extracted from the mid-panel region. However, the distress noted in the cores from Highway 146 was not as severe as the distress observed in cores from US 63. Petrographic examination of the cores indicated that the cracking was again related to a failure of the coarse aggregate particles (see Appendix B). This is consistent with "normal" behavior for coarse aggregate from Malcom mine. Note that the concrete in US 63 is about three years older than the concrete in Highway 146; this may partially explain the difference in distress noted at the two sites.

The GPR survey conducted by MGPS, Inc., was about four-tenths of a mile in length. The data was collected while moving with traffic (speed varied from about 15 to 40 mph). The responses observed in the Highway 146 data are very similar to those in the US 63 southbound data that were presented earlier. However, there are fewer observable anomalies. This is in good agreement with the distress noted in the cores. The response that appears to come from dowel chairs is found in both the 900 and 400 MHz data (see Figure 21). The vertical red hash mark denotes the beginning of the data collection; this corresponds to the location of a manhole at mile marker 20 on Highway 146.

Figure 18. Overview of Hwy 146 Pavement Site (looking north, note patches)

Figure 19. Common Distress Features at Hwy 146, Spalling or Corner Breaks

Figure 20. Cores Extracted from Hwy 146 (note deterioration at base of cores)

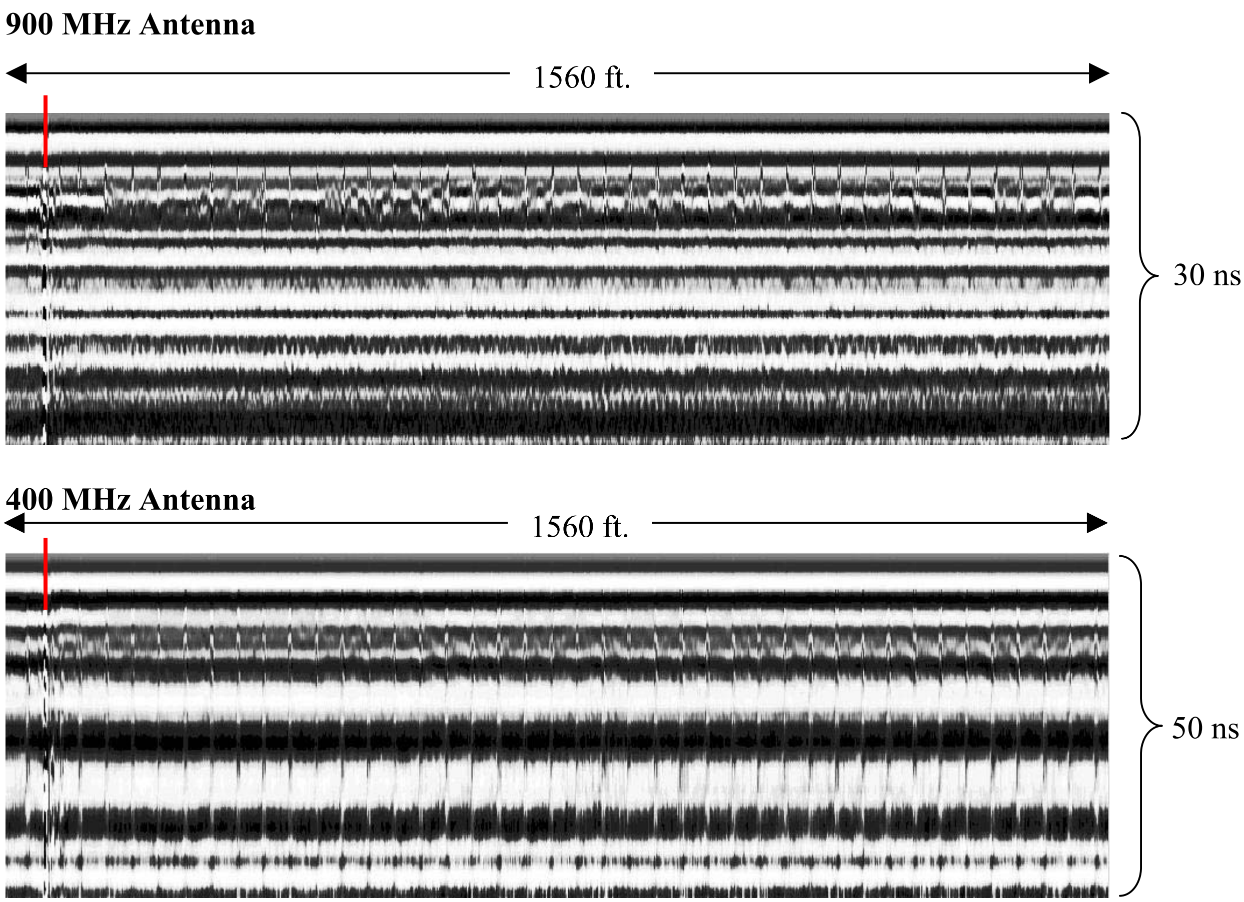


Figure 21. GPR Data from Hwy 146 Northbound

P 73, Webster County

County road P 73 is a two-lane PCC pavement that was constructed as a stub to US 20. The stub is about 300 feet long, and it ties into an asphalt road that goes north into the town of Duncombe (see Figure 22). Coarse aggregate from Fort Dodge mine was used in the pavement. This particular coarse aggregate is a crushed limestone that has at least a Class 3 durability rating, and hence, should show no distress after 30 years of service. The GPR site was limited to the stub of concrete pavement. Only the southbound lane of the pavement was evaluated in this study. Figure 22 also illustrates the paint markings where cores were to be extracted for pavement thickness and petrographic examination.

P 73 exhibits extensive cracking over most of the pavement slab. Typical cracking patterns that were observed are shown Figure 23. Cores extracted from the pavement exhibited significant cracking (see Figure 24). The core extracted from the pavement joint split into pieces during extraction. Petrographic examination of the cores indicated that the cracking was not related to the coarse or fine aggregate used in the mix (see Appendix C). Instead, the distress appeared to be concentrated in the paste fraction of the concrete.

GPR testing conducted by MGPS, Inc., traversed a short section of pavement that was 298 feet long. The data were collected while moving slowly (about 10 to 20mph). Significant cracking and subsurface distress dominates the P 73 pavement section that was surveyed. It is interesting that the response from the high frequency 1.5 GHz antenna does not appear to image this distress (since this response is flat and consistent along the length of the site, see Figure 25). However, the irregular features in the 400 MHz response indicate that the pavement subgrade may not be providing continuous support or that there may be moisture gradients beneath the surface of the pavement. The unusual radar response this concrete produced, in light of the known distress modes, indicated that the radar response may have been masked by excessive moisture in the pavement or subgrade. It is also possible that the radar equipment did not perform optimally at this site. Whatever the reason, it appears that a more thorough survey of this site will be required to properly identify the problems that exist here using GPR.

The GPR survey conducted by ISU personnel produced information in good agreement with that obtained from the MGPS, Inc., survey. The ISU survey had a length of about 75 to 150 feet (25 to 45 meters, depending on the antenna that was used) and was collected at a maximum speed of 5 mph. The typical feature noted in the test results was a flat and consistent response along the length of the survey (see Figure 26). Hence, it seems unlikely that the radar equipment did not perform adequately on two separate studies. Rather, it may not be possible to image this type of materials-related distress using GPR. Additional work will be needed to clarify why GPR does not work at this site.

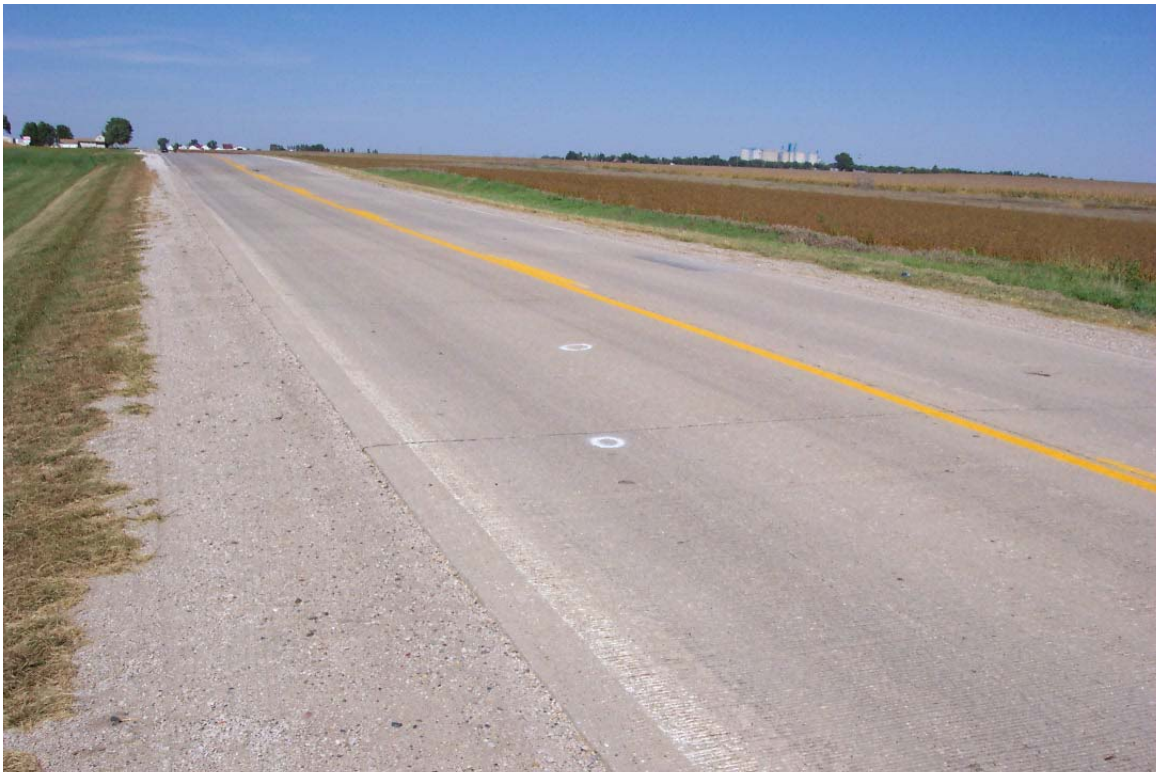


Figure 22. Overview of the GPR Site for the P 73 Pavement Site (looking north)

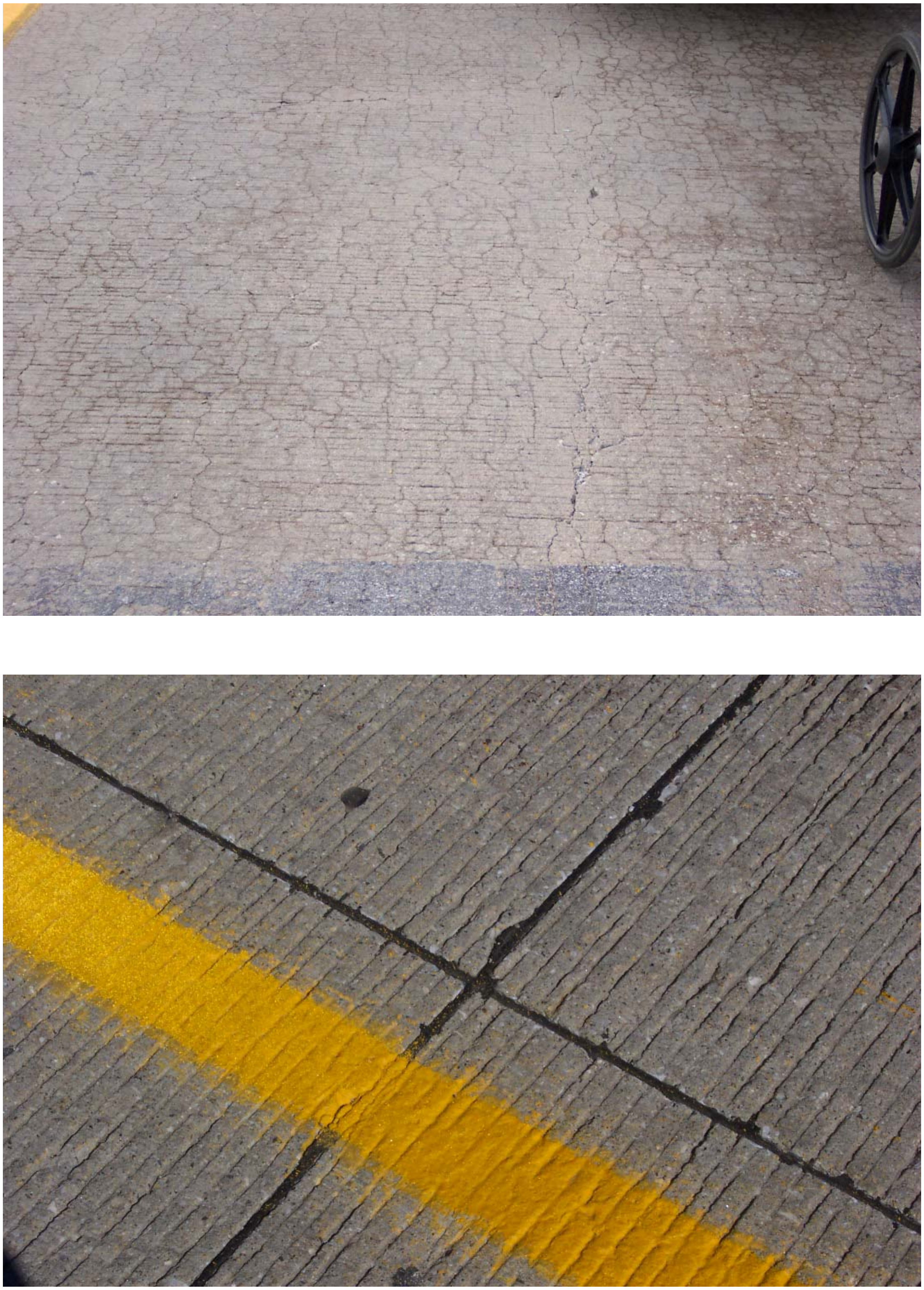


Figure 23. Distress Noted on P 73
(top, map cracking over much of slab; bottom, not distinct at joints)

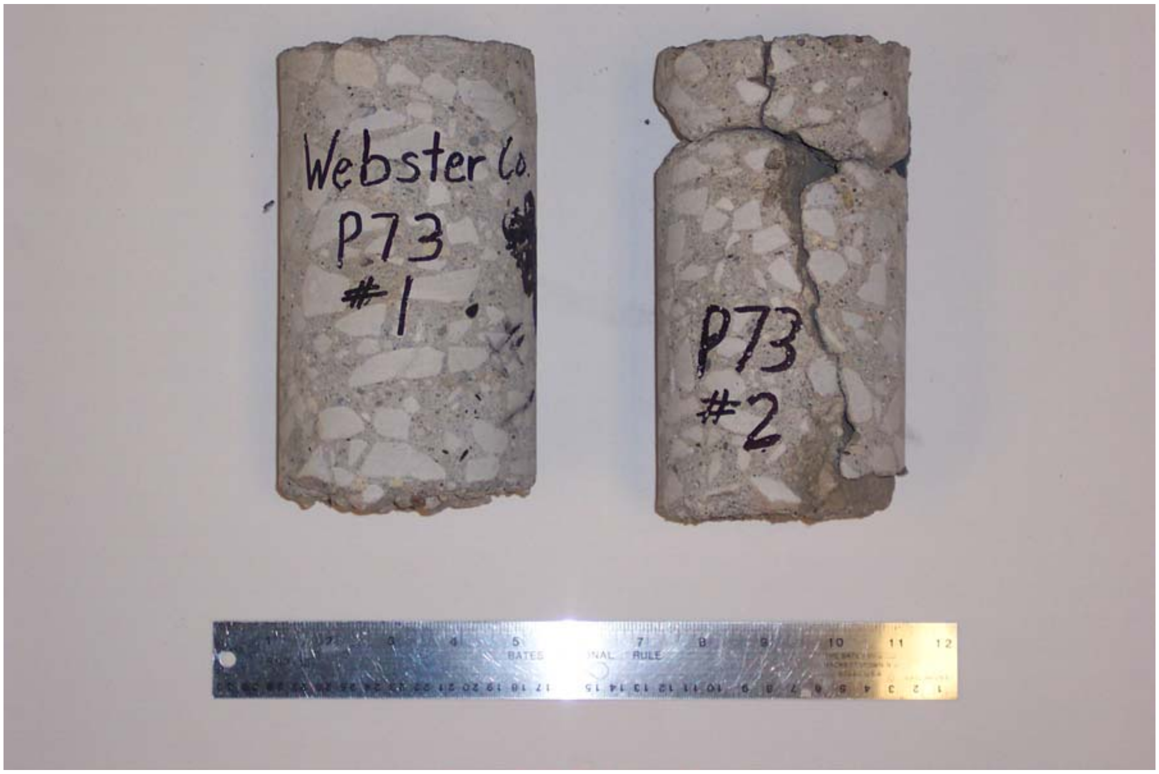
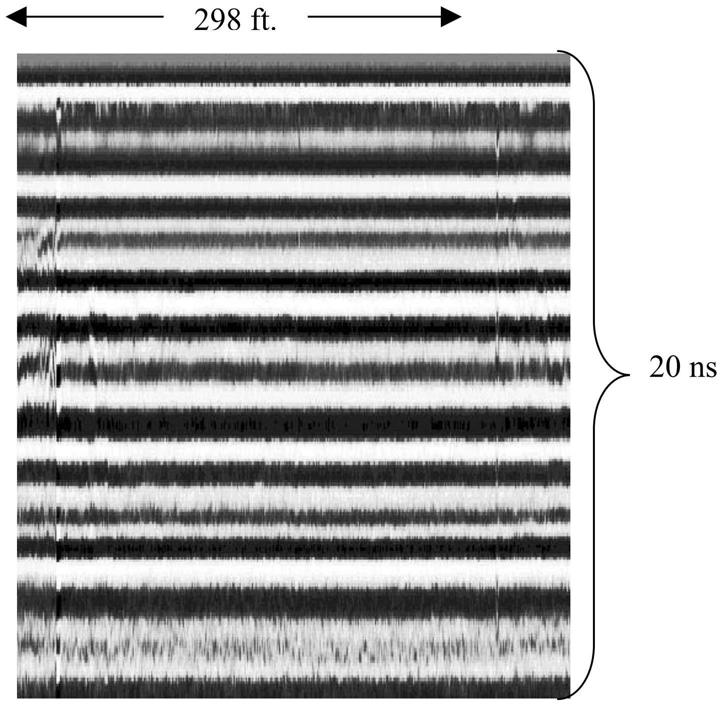


Figure 24. Cores Extracted from the P 73 Pavement

1500 MHz Antenna



400 MHz Antenna

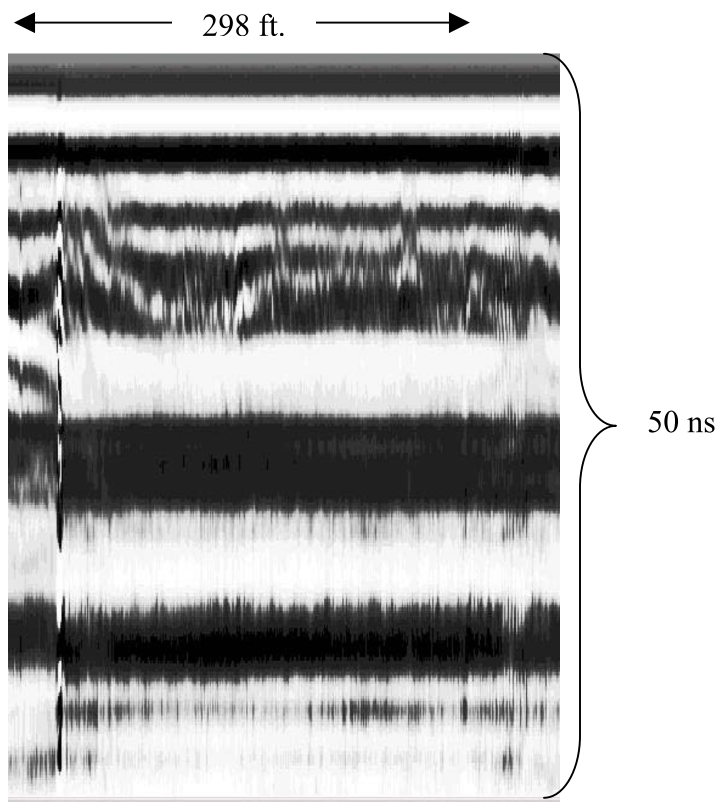
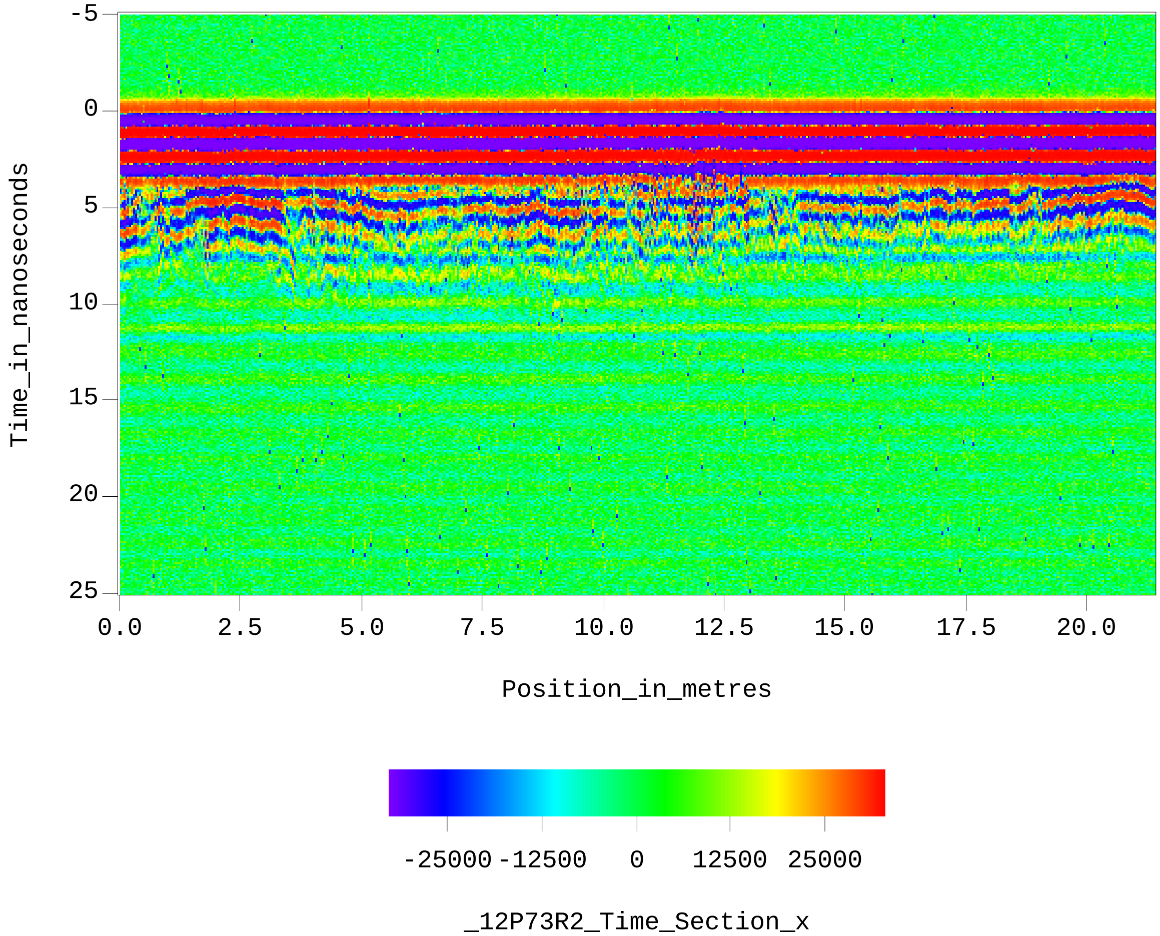


Figure 25. GPR Data from P 73, Southbound

1200 MHz Antenna



900 MHz Antenna

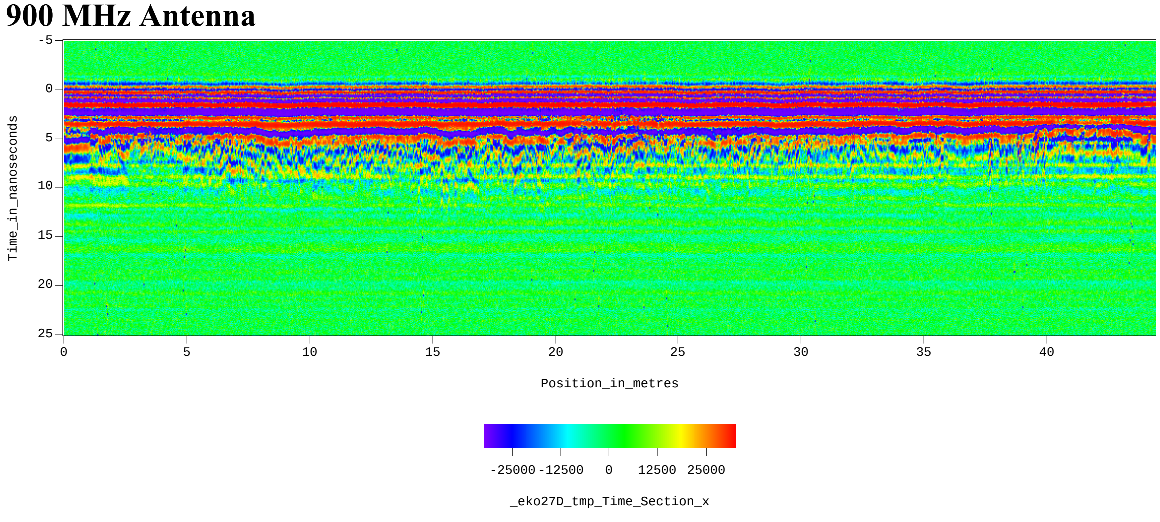


Figure 26. High-Resolution GPR Data from P 73 (note lack of distress features in signal)

Hwy 175, Hamilton County

Highway 175 is a two-lane PCC pavement that was constructed with a design thickness of 8.0 inches. The roadway runs between Jewell and Ellsworth, Iowa, and was constructed as a research test section to evaluate the use of fly ash in portland cement concrete pavements. The roadway is now 23 years old and exhibits very little evidence of any type of distress. A typical section of the roadway is illustrated in Figure 27. Coarse aggregate from Moberly mine was used in the pavement. This particular coarse aggregate is a crushed limestone that should show no distress after 30 years of service. This particular section of road was included in the study to act as a control section. MGPS, Inc., conducted a GPR survey from milepost 156.2 to milepost 158.3 (actually ending at the bridge on the east side of the site, just before the town of Ellsworth). Only the eastbound lane of the pavement was evaluated in this study. For the sake of brevity, the results of the ISU GPR study for this site will not be presented in this report (also equipment problems were experienced at this particular site).

Very few cracks were noted on the section of Highway 175 that was evaluated for this study. Some aggregates were exposed on the pavement surface due to wear; however, the depth of abrasion did not appear excessive for a road with nearly 25 years of service. Cores extracted from the pavement were intact and sound (see Figure 28). Petrographic examination of the cores indicated that some particles of fine aggregate (shale and occasionally chert) exhibited minor cracking. Also, the core from the transverse joint from the first test section (near milepost 156.2) did contain some cracked coarse aggregate particles (see Appendix D).

GPR testing conducted over this roadway traversed nearly two miles of pavement, including three contiguous test sites. These sites were used in the study as control sections. The data were collected while moving at highway speed (about 55 mph). The radar response from all three of the antennas that were used for the GPR survey produced a flat, consistent response throughout the majority of the survey. An exception to this characteristic response occurred at the location of a drainage culvert that passes underneath the road near the western end of the site. The red encircled locations in the data presented in Figure 29 indicate the response to the drainage culvert based on the expected site location for this feature. The remaining data are essentially free from anomalies, which are representative of this site.

Figure 27. Overview of Hwy 175 Pavement Site (looking east)

Figure 28. Typical Cores Extracted from Hwy 175 (eight cores were taken from the site)

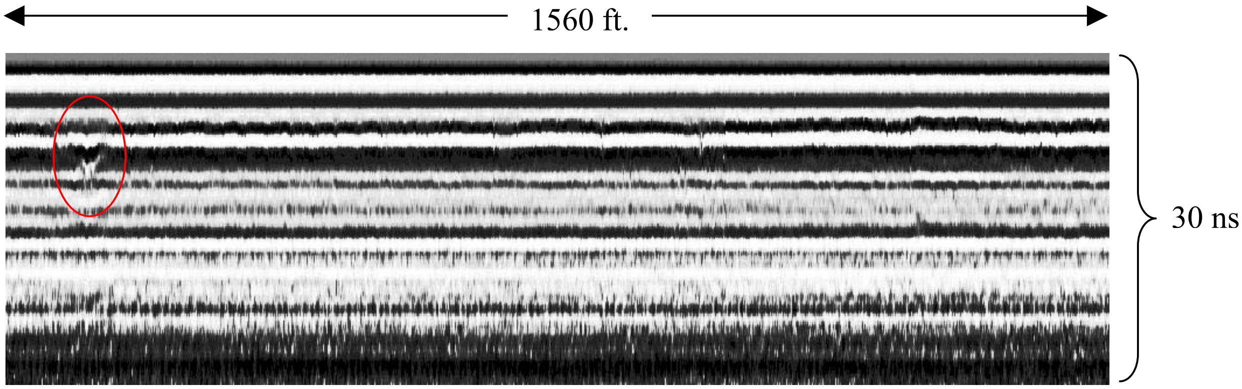


Figure 29. GPR Data from Hwy 175 Eastbound (900 MHz antenna)

SUMMARY

In summary, a literature survey was conducted to evaluate potential nondestructive test methods that could be used to detect materials-related distress in portland cement concrete pavements. The various nondestructive test methods were then subjected to selection criteria that helped to reduce the size of the list so that specific techniques could be investigated in more detail. The main test methods that were determined to be applicable to this study included stress-wave propagation techniques (impact-echo and spectral analysis of surface waves techniques), infrared thermography, ground penetrating radar and visual inspection. The ground penetrating radar (GPR) technique was selected for a preliminary round of “proof-of-concept” trials.

GPR surveys were carried out over a variety of portland cement concrete pavements for this study using two different systems. One of the systems was a state of the art GPR system that allowed data to be collected at highway speeds. The second system was a less sophisticated system that was commercially available. Surveys conducted with both sets of equipment have produced test results capable of identifying subsurface distress in two of the three sites that exhibited internal cracking due to MRD. Both systems failed to detect distress in a single pavement that exhibited extensive cracking. Both systems correctly indicated that the control pavement exhibited negligible evidence of distress. The initial positive results presented here indicate that a more thorough study (incorporating refinements to the system, data collection, and analysis) is needed. Improvements in the results will be dependent upon defining the optimum number and arrangement of GPR antennas to detect the most common problems in Iowa pavements. In addition, refining high-frequency antenna response characteristics will be a crucial step toward providing an optimum GPR system for Iowa pavements.

Experience with the two different GPR systems and different software packages used in Phase 1 indicated that proper vendor selection is very important. Typically it was difficult to interchange data between the various programs. Also, each set of programs tended to have their own strengths and weaknesses. Hence, the preliminary study was restricted by not being able to analyze data using either set of programs. This must be avoided in future work because one must be able to analyze and present data in as flexible manner as is possible.

PHASE 2

Phase 2 Goals and Objectives

The overall goal of this project is to research the use of nondestructive test methods to simplify the in-situ detection of materials-related distress in portland cement concrete pavements. The test results obtained from Phase 1 indicated that additional work is needed to study the ability of GPR to detect materials-related distress. As mentioned earlier, the GPR technique was successful at identifying aggregate-induced cracking (frost attack or durability cracking) in two of the pavement sites in this study. This was accomplished even though the symptoms (i.e., cracking and staining) were not readily evident at the surface of the pavement. Hence, the GPR technique contributed information about distress that was not readily available from visual inspection. However, the GPR study failed to identify any distress in a severely map-cracked pavement site that had been diagnosed with premature distress. This suggests that a more thorough study of these various pavement sites is needed to help explain why the technique fails to detect cracking related to premature distress. Also, difficulties related to firmware and software development for the Mandli pavement profile scanner have been resolved and that system is now available for field testing.

The specific objectives for Phase 2 include the following:

- Evaluation of how GPR antenna type, configuration, and survey vehicle speed influence the detection of materials-related distress.
- Re-evaluation of the P 73 test site (premature distress cracking) to determine why the GPR method failed to indicate distress. Special attention will be given to site variables that influence GPR response (i.e., moisture content or the presence of deicer salts) and may have obscured the signal from the slab.
- Evaluation of the PPS-photolog system to see if it can be used to effectively locate and quantify surface distress.
- Evaluate the feasibility of combining the results from the GPR, PPS, and photolog systems to simplify the visualization of pavement distress features.

Phase 2 Work Plan and Task Schedule

The specific work plan (Table 5) and task schedule (Table 6) for Phase 2 are given below.

Table 5. Work Plan

Task	Description	Completion Date (Expected/Actual)	% of Task Completed
1	Contact project consultants to • review project goals • provide Phase 1 test results • identify research and equipment needs for Phase 2	Mar 2003/Mar 2003	100%
2	Select pavement sites that will be evaluated during the field study.	Apr 2003/June 2003	100%
3	Request price quotes on ground penetrating radar equipment and accessories needed for Phase 2. Order the equipment and software that best meets the needs of the research project.	May 2003/	50%
4	Contact Mandli and the Iowa DOT to arrange the use of the pavement profile scanner and photolog system van.	May 2003/May 2003	100%
5	Conduct "proof of concept" field studies. Analyze test results and compare to conventional distress evaluation techniques.	Oct 2003/	30%
6	Prepare the final report.	Dec 2003/	0%

Table 6. Task Schedule

Task	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec
1	XX	XX	XX									
2			XX	XX								
3				XX	XX							
4					XX							
5						XX	XX	XX	XX	XX		
6										XX	XX	XX

REFERENCES

1. Hansen, W.C. Uniformity of Cores an Indication of Pavement Quality. *Proceedings of the Highway Research Board*, Vol. 20, 1940, pp. 568–586.

2. Van Dam, T.J., L.L. Sutter, K.D. Smith, M.J. Wade, and K.R. Peterson. *Guidelines for Detection, Analysis, and Treatment of Materials-Related Distress in Concrete Pavements*. Vols. 1, 2, and 3. Report FHWA-RD-01-163. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., 2002.

3. Shekharan, A.R., G.R. Rada, G.E. Elkins, and W.Y. Bellinger. Assessment of Long-Term Pavement Performance Plan Wall Projection-based Distress Data Variability. *Transportation Research Record*, No. 1643, 1998, pp. 95–98.

4. Goodman, S.N. Assessing the Variability of Surface Distress Surveys in Canadian Long-Term Pavement Performance Program. *Transportation Research Record*, No. 1764, 2001, pp. 112–118.

5. Concrete and Aggregates. *Annual Book of ASTM Standards*. Vol. 4.02. ASTM C 856. American Society for Testing and Materials, West Conshohocken, Pa., 2002.

6. Raj, B., T. Jayakumar, and M. Thavasimutha. *Practical Non-Destructive Testing*. 2nd edition. Narosa Publishing House, New Dehli, India, 2002.

7. American Concrete Institute. *Manual of Concrete Practice.* Part 2. ACI 228.2R-98. American Concrete Institute, Farminton Hill, Mich., 2002.

8. Malhotra, V.M., and N.J. Carino, editors. *CRC Handbook on Nondestructive Testing of Concrete*. CRC Press, New York, 1991.

9. Malhotra, V.M. Nondestructive Tests. *ASTM STP 169C*. P. Klieger and J.F. Lamond, editors. American Society for Testing and Materials, West Conshohocken, Pa., 1994, pp 320–338.

10. Carino, N.J. Recent Developments in Nondestructive Testing of Concrete. *Advances in Concrete Technology.* V.M. Malhotra, editor. MSL 92-6(R). Canada Communications Group Publishing, Ottawa, Canada, 1992, pp. 281–328.

11. Whitehurst, E.A., and W.E. Parker. Dynamic Tests. *ASTM STP 169*. American Society for Testing and Materials, Philadelphia, Pa., 1956, pp. 104–114.

12. Whitehurst, E.A., and W.E. Parker. Dynamic Tests. *ASTM STP 169A*. American Society for Testing and Materials, Philadelphia, Pa., 1966, pp. 176–188.

13. Whitehurst, E.A., and V.M. Malhotra. Nondestructive Tests. *ASTM STP 169B*. American Society for Testing and Materials, Philadelphia, Pa., 1978, pp. 200–225.

14. McCall, B., and J. Whited. *Demonstration Project 85, Video Imaging Systems for Highway Applications.* FHWA-DP-90-085-004. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., May 1990.

15. Zimmerman, K.A., and R. Knox. Improving IDOT's Pavement Rating Process. *Journal of Infrastructure Systems*, Vol. 4, No. 2, 1998, pp. 79–85.

16. Sansalone M., and N. J. Carino. Stress Wave Propagation Methods. *CRC Handbook on Nondestructive Testing of Concrete*. V.M. Malhotra and N.J. Carino, editors. CRC Press, New York, 1991, pp. 275–304.

17. Concrete and Aggregates. *Annual Book of ASTM Standards*. Vol. 4.02, ASTM C 1383. American Society for Testing and Materials, West Conshohocken, Pa., 2002.

18. Nazarian, S., D. Yuan, and M.R. Baker. Use of Seismic Pavement Analyzer for NDT of Road. *Nondestructive Testing and Evaluation of Infrastructure*, H.D. Reis and B.B. Djordjevic, editors. American Society for Nondestructive Testing, Inc., Columbus, Ohio, 1998, pp. 121–132.

19. Carino, N.J. Nondestructive Test Methods in Concrete Construction Engineering Handbook, E. Nawy, editor. CRC Press, New York, 1997, Chapter 19.

20. Road and Paving Materials; Vehicle-Pavement Systems. *Annual Book of ASTM Standards*. Vol. 4.03. ASTM D 4788. American Society for Testing and Materials, West Conshohocken, Pa., 2002.

21. Weil, G.J. Infrared Thermographic Techniques. *CRC Handbook on Nondestructive Testing of Concrete*. V.M. Malhotra and N.J. Carino, editors. CRC Press, New York, 1991, pp. 305–316.

22. Aperio Limited. High Speed Information from Highway Networks. www.aperio.co.uk. Accessed March 2003.

23. Clemena, G.C. Short-Pulse Radar Methods. *CRC Handbook on Nondestructive Testing of Concrete*. V.M. Malhotra and N.J. Carino, editors. CRC Press, New York, 1991, pp. 253–274.

24. Road and Paving Materials; Vehicle-Pavement Systems. *Annual Book of ASTM Standards*. Vol. 4.03. ASTM D 4748. American Society for Testing and Materials, West Conshohocken, Pa., 2002.

25. Barnes, C.L., and J.F. Trottier. Ground-penetrating Radar for Network-Level Concrete Deck Repair Management. *Journal of Transportation Engineering*, Vol. 126, No. 3, 2000, pp. 257–262.

26. Scott, M., N. Davidson, J.C. Duke, Jr., M. Moore, G. Washer, and R. Weyers. Evaluating the PERES Ground Penetrating Radar System for Bridge Deck Inspection Using Image Processing and Pattern Recognition Data Analysis. *Structural Materials Technology IV*. Technomic Publishing Co., Lancaster Pa., 2000, pp. 61–66.

27. Ground Penetrating Radar Identifies Pavement Segments. *Public Works*, Vol. 122, No. 1, January 1991, pp. 66–67.

28. Fernando, E.G., K.R. Maser, and B. Dietrich. Implementation of Ground Penetrating Radar of Network Level Pavement Evaluation in Florida. *Proceedings of the Fifth International Conference on Ground Penetrating Radar*, Vol. 1, 1994, pp. 351–365.

29. Adams, G., N. Anderson, J. Baker, M. Shoemaker, and A. Hatheway. Ground-Penetrating Radar Study of Subsidence Along US Interstate 44 Springfield, Missouri. *Proceedings of the Seventh International Conference on Ground Penetrating Radar*, Vol. 2, 1998, pp. 619–623.

30. Hickman, S., S. Cardimona, D. Webb, T. Lippincott, J. Wenzlick, and N. Anderson. Ground Penetrating Radar Survey of Interstate 70 Across Missouri. *GPR 2000*, December 2000. www.fhwa.dot.gov/bridge/geophabs.htm. Accessed February 2002.

31. Saarenketo, T., and T. Scullion. *Ground Penetrating Radar Applications on Roads and Highways*. Research Report 1923-F. Texas Transportation Institute, College Station, Tex., 1994.

32. Scullion , T., and T. Saarenketo. Application of Ground Penetrating Radar for Network and Project Level Pavement Management Survey Systems. *Proceedings of the Fourth International Conference on Managing Pavements*, Vol. 1, 1998, pp. 254–268.

33. Smith, S.S., and T. Scullion. *Development of Ground-Penetrating Radar Equipment for Detecting Pavement Condition for Preventative Maintenance*. Report SHRP-H-672. Strategic Highway Research Program, National Research Council, Washington, D.C., 1993.

34. Maser, K.R. Pavement Characterization Using Ground Penetrating Radar: State of the Art and Current Practice. *Nondestructive Testing of Pavements and Backcalculation Moduli*. Vol. III. ASTM STP 1375. S.D. Tayabji and E.O. Lukanen, editors. American Society for Testing and Materials, West Conshohocken, Pa., 2000, pp. 22–27.

35. Romero, F.A., D. Allen, R.L.Roberts, and J. Fleckenstein. Ground-Penetrating Radar Investigation of and Interstate Concrete Pavement for Evidence of Segregation Caused by Poor Consolidation During Construction. *GPR 2000*, December 2000. www.fhwa.dot.gov/bridge/geophabs.htm. Accessed February 2002.

36. Federal Communications Commission. FCC 02-48, Revision of Part 15 of the Commission's Rules Regarding Ultra-Wideband Transmission Systems. Released April 22, 2002.

37. Sensors and Software. FCC Related Issues. www.sensoft.ca/news/index.htm. Accessed February 2003.

38. Gros, X.E. Multisensor Data Fusion and Integration in NDT. *Applications of NDT Data Fusion*. Kluwer Academic Publishers, Norwell, Mass., 2001, pp. 1–12.

39. Phoenix Scientific, Inc. www.phnx-sci.com/pages/scantech.html. Accessed January 2003.

40. Mandli Communications, Inc. Digilog Pavement Systems. www.mandli.com/products/pavement.htm#pps. Accessed January 2003.

41 McElwee, W.D., and J.W. Eales. *A Non-destructive Method for Determining the Thickness of Sound Concrete on Older Pavements*. Final Report HR-250. Iowa Highway Research Board, Iowa Department of Transportation, Ames, Iowa, December 1983.

APPENDIX A: SUMMARY AND GENERAL OBSERVATIONS OF THE CORES EXTRACTED FROM US 63

Table A.1. Summary of Cores from US 63

	Core 6	Core 7	Core 8	Core 9
Location	Joint	Joint	Mid-panel	Joint
Length (inches)	7.25	9.0	9.5	7.75

Distress was the results of fractures occurring at the base of the slab. The fractures were sub-parallel to the base. Distress was worse at the joints than the mid-panel, although the core from the mid-panel showed some distress in the form of fractured coarse aggregate particles. Fractures were seen at the paste/aggregate interface and in the interior of the aggregate particles. Some of the fractures terminated in the paste. The paste was discolored around some of the coarse aggregate particles. In the coarse aggregate particles, both coarse and fine-grained particles contained fractures. There was no indication of alkali silica reactions (ASR) or alkali carbonate reactions. The most likely cause for the distress was failure of the coarse aggregate particles, possibly due to the capillary pore system. The distress may have been magnified due to poor drainability of the subbase and possibly due to anti-icing brines leaking through the joints.

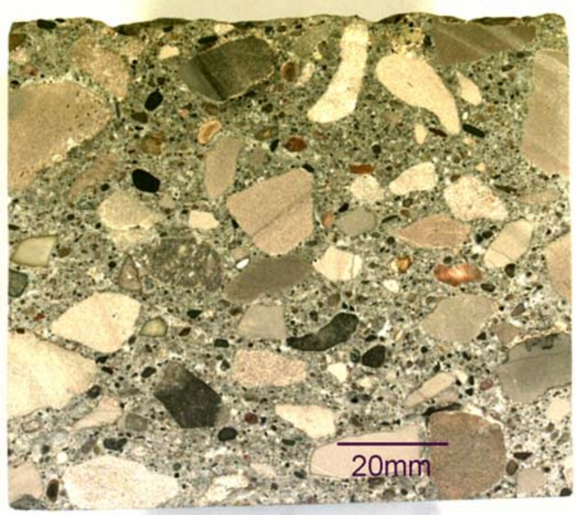


Figure A.1. Top of Core from Near a Joint

Figure A.2. Top of Core from the Mid-panel

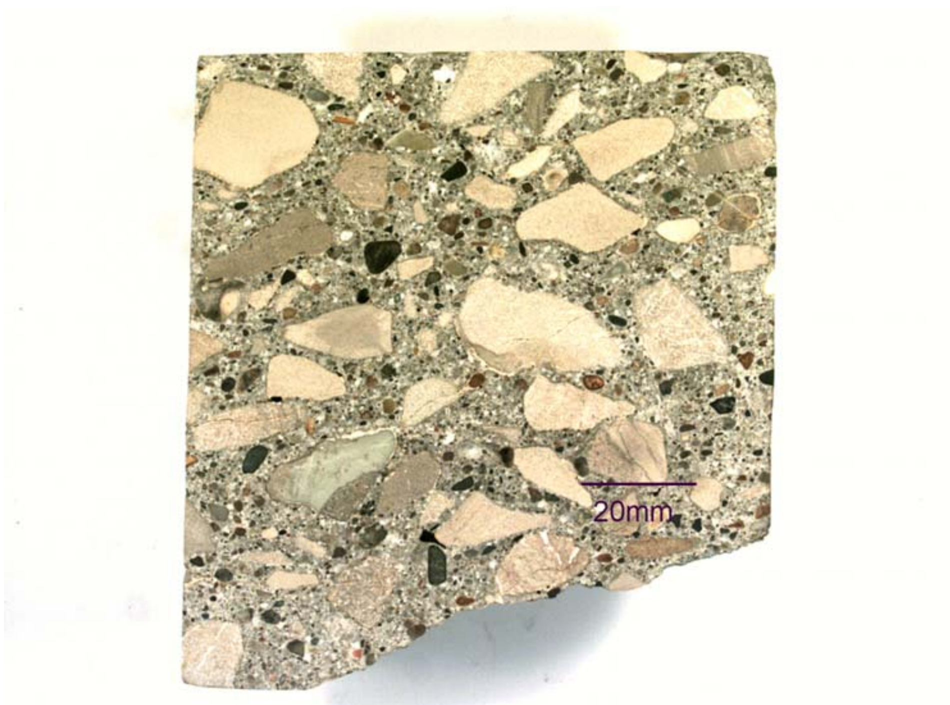


Figure A.3. Bottom of Core Near the Joint

Figure A.4. Bottom of Core Near the Mid-panel

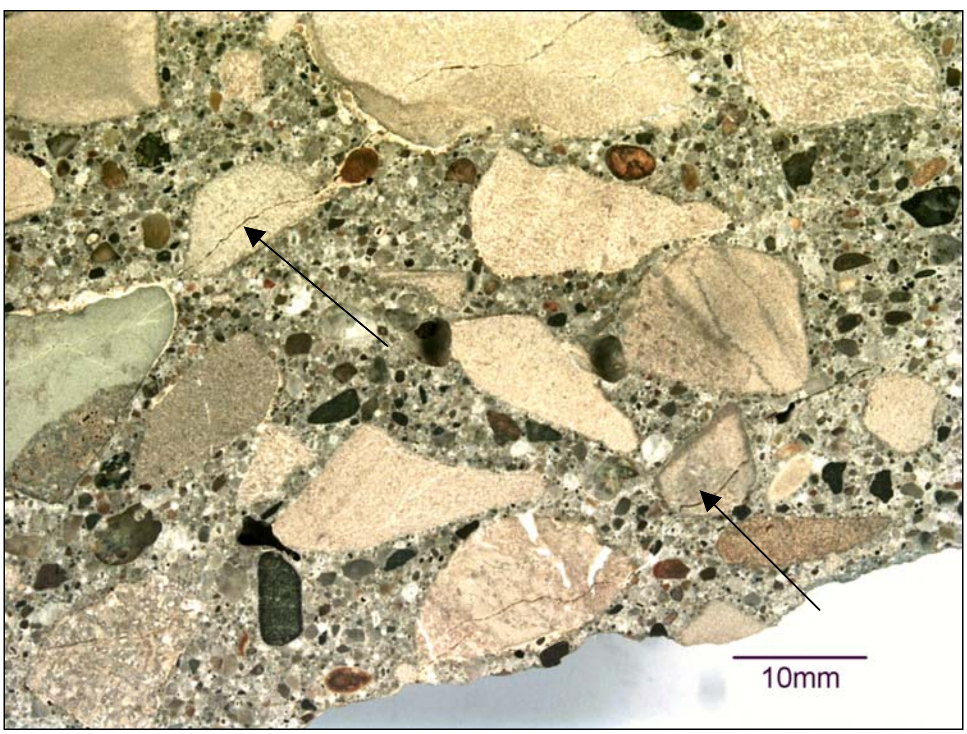


Figure A.5. Base of core from near a joint shows sub-parallel fractures also parallel the base of the slab.

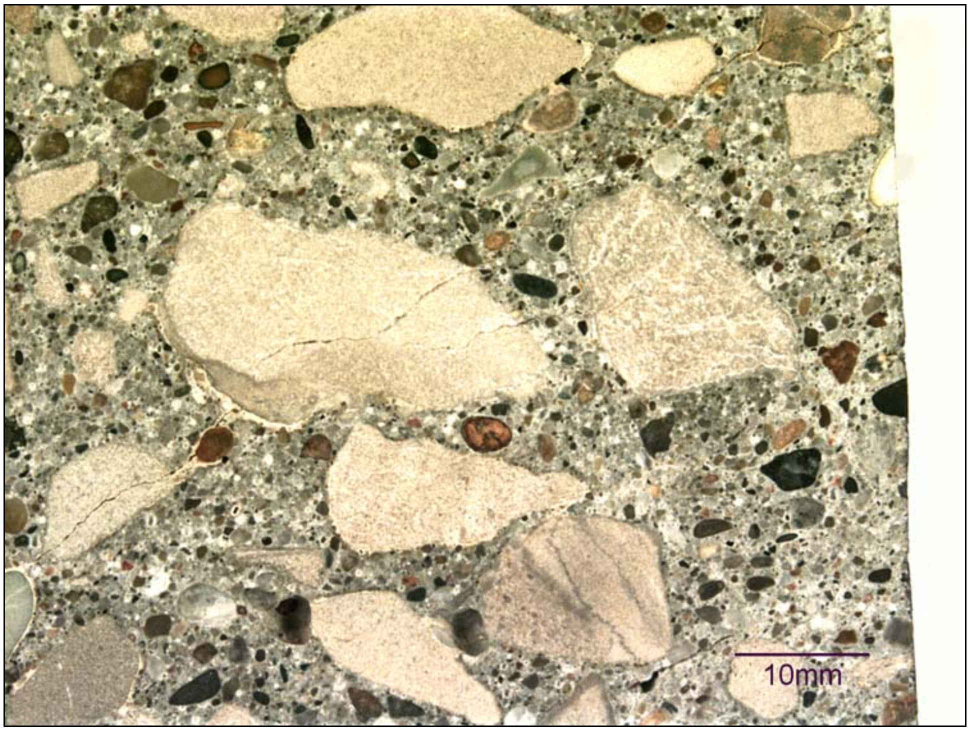


Figure A.6. Most of the fractures were through the interior of the coarse aggregate particles. Fractures can be traced through multiple aggregate particles.

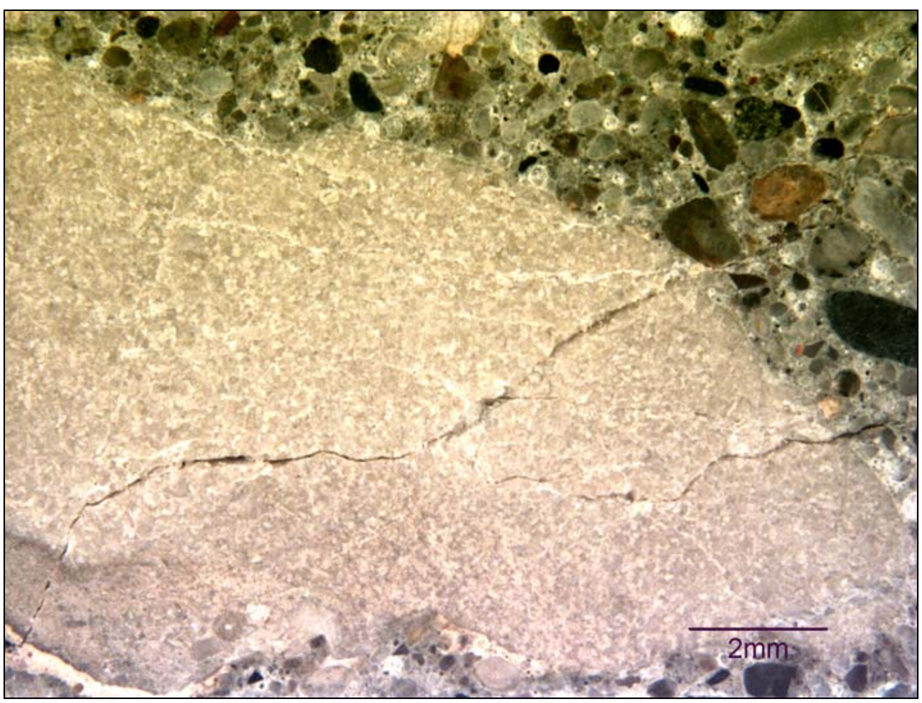


Figure A.7. Fracture pattern in a coarse aggregate particle in the bottom of the core from near the joint.

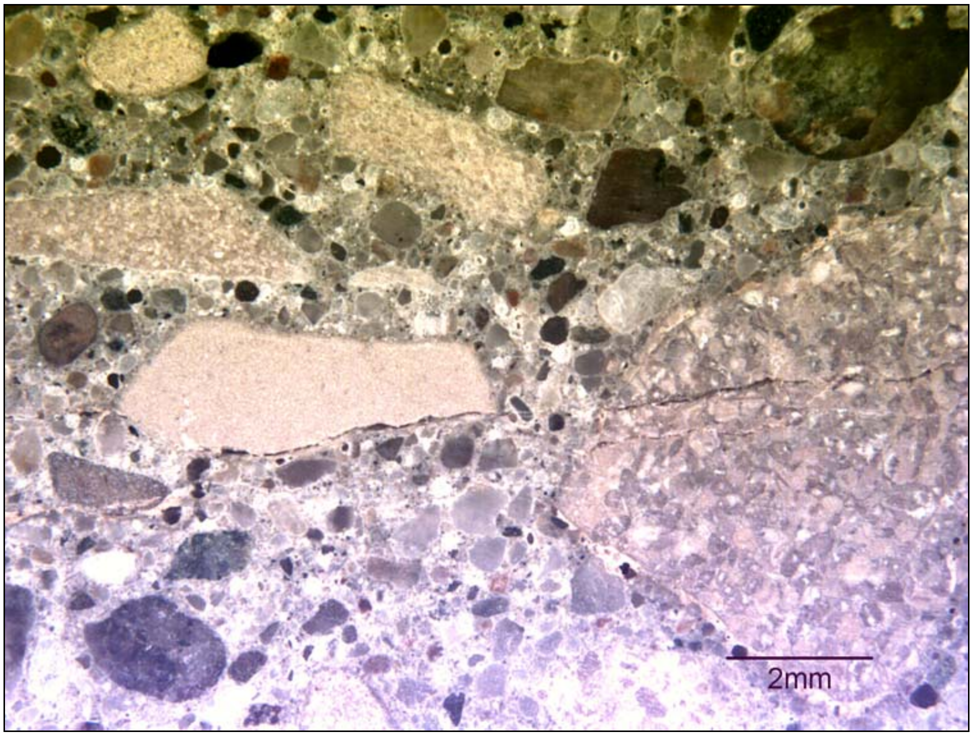


Figure A.8. Some fractures were at the aggregate paste interface and others passed through the interior of the aggregate (bottom of the core near joint).

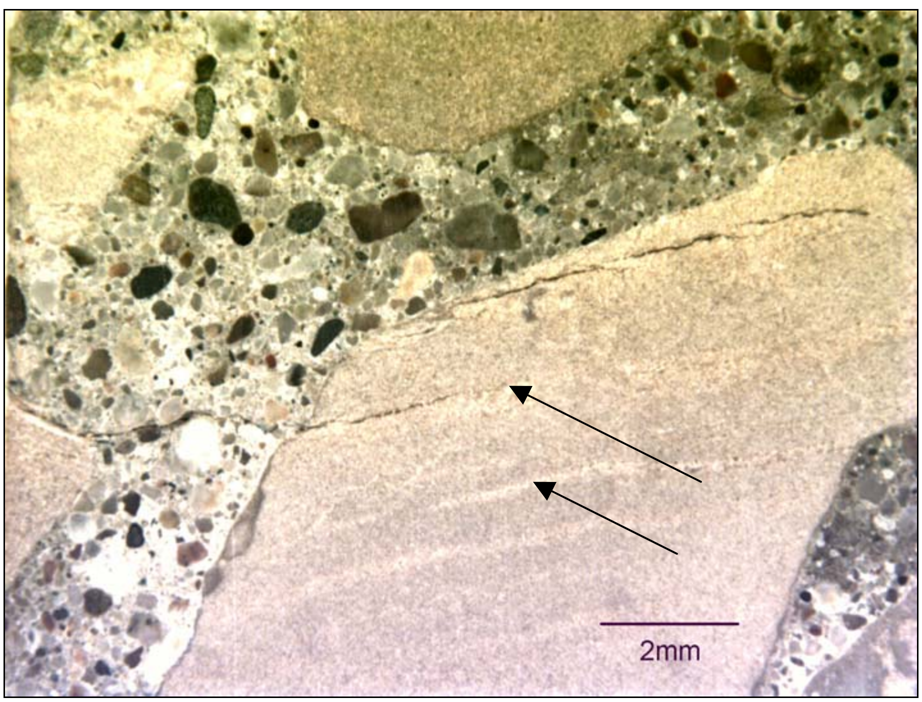


Figure A.9. Light colored bands in this coarse aggregate particle were zones of weakness that resulted in fractures.

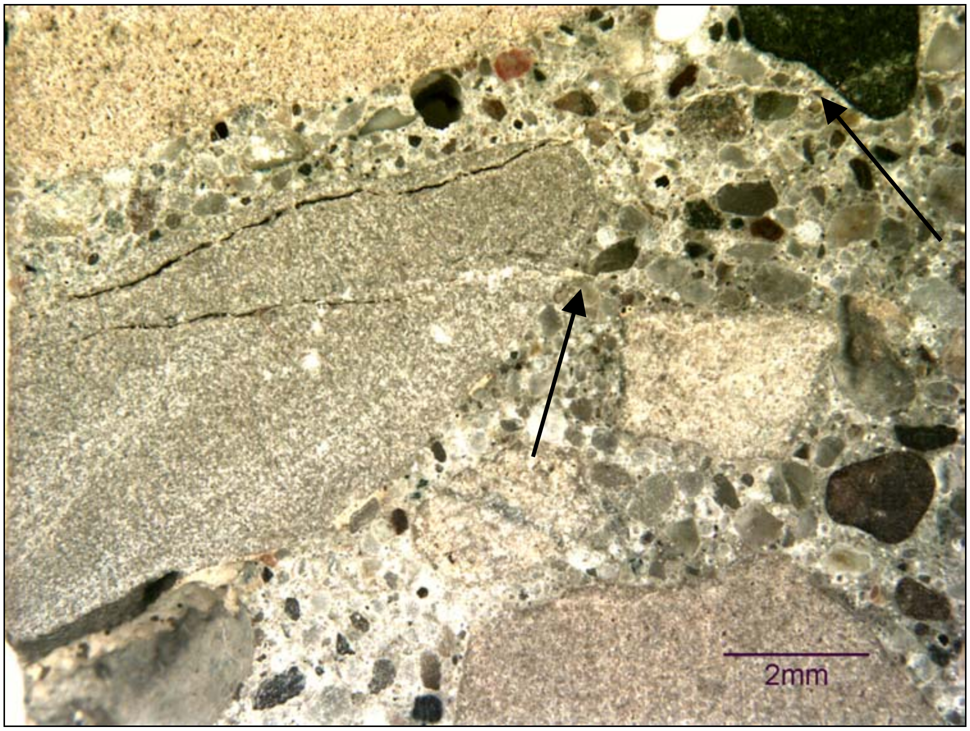


Figure A.10. Some fractures extended into the paste and others terminated within the paste.

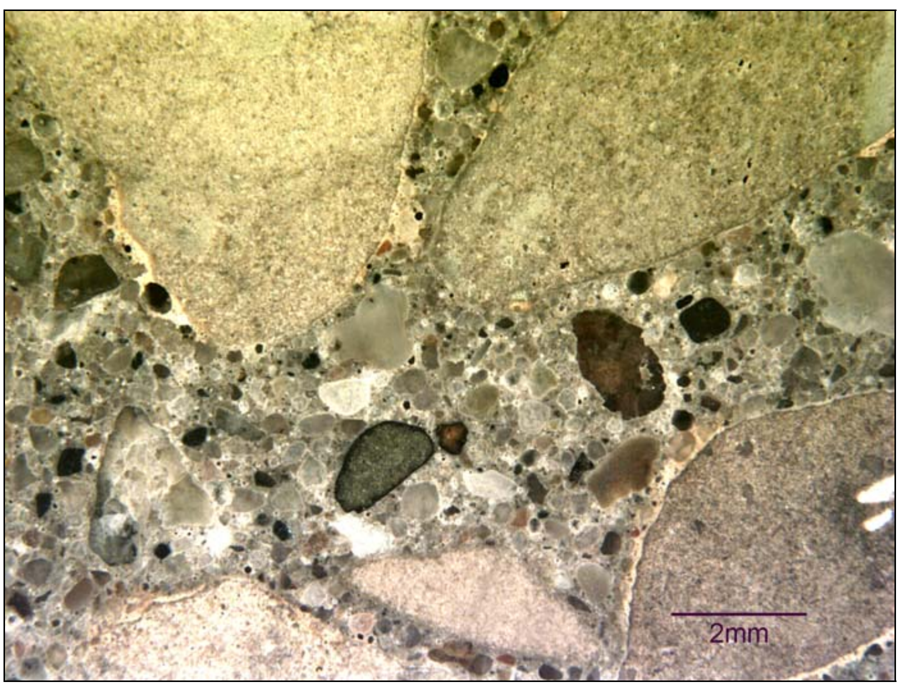


Figure A.11. Discoloration of the paste at the paste aggregate interface. Core was from the bottom of the mid-panel.

APPENDIX B: SUMMARY AND GENERAL OBSERVATIONS OF THE CORES EXTRACTED FROM HWY 146

Table B.1. Summary of Cores from Hwy 146

	Core 1	Core 2	Core 3	Core 4	Core 5
Location	Joint	Joint	Joint	Joint	Mid-panel
Length (inches)	9.25	9.25	8.25	9.0	9.25

Distress was the result of fractures occurring at the base of the slab. The distress was very similar to that observed in US 63. The paste was discolored around some of the coarse aggregate particles. In the coarse aggregate particles, both coarse and fine-grained particles contained fractures. The most likely cause for the distress was failure of the coarse aggregate particles, possibly due to the capillary pore system.

Figure B.1. Bottom of Core 3 (joint)

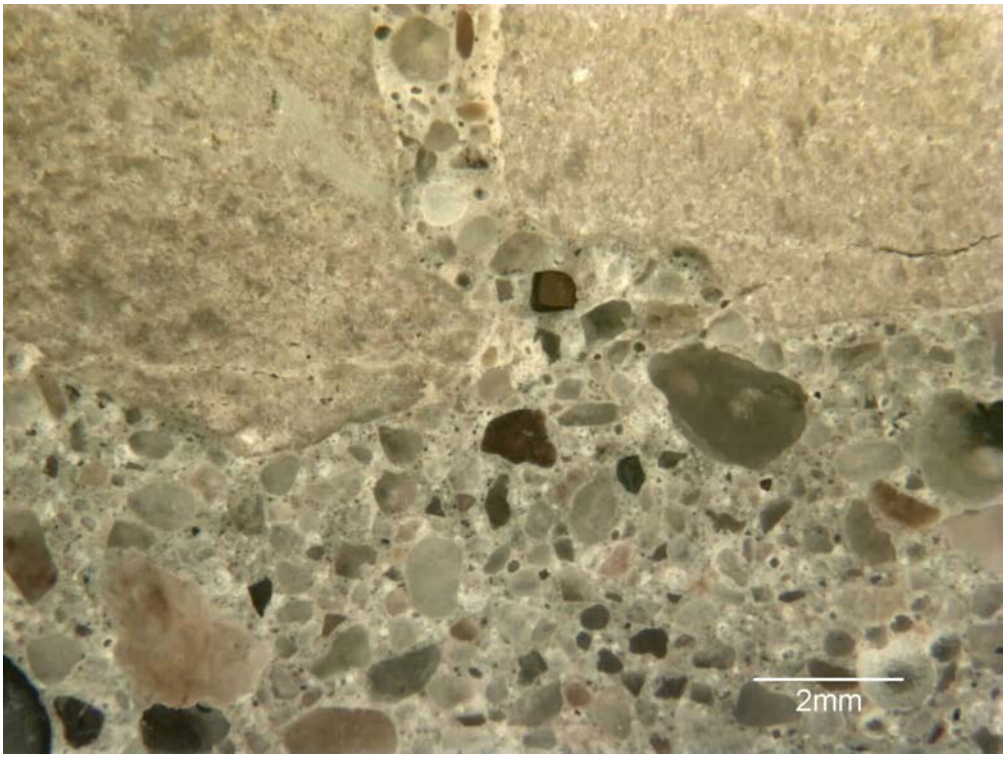


Figure B.2. Core 3: note crack in coarse aggregate particle terminating in paste

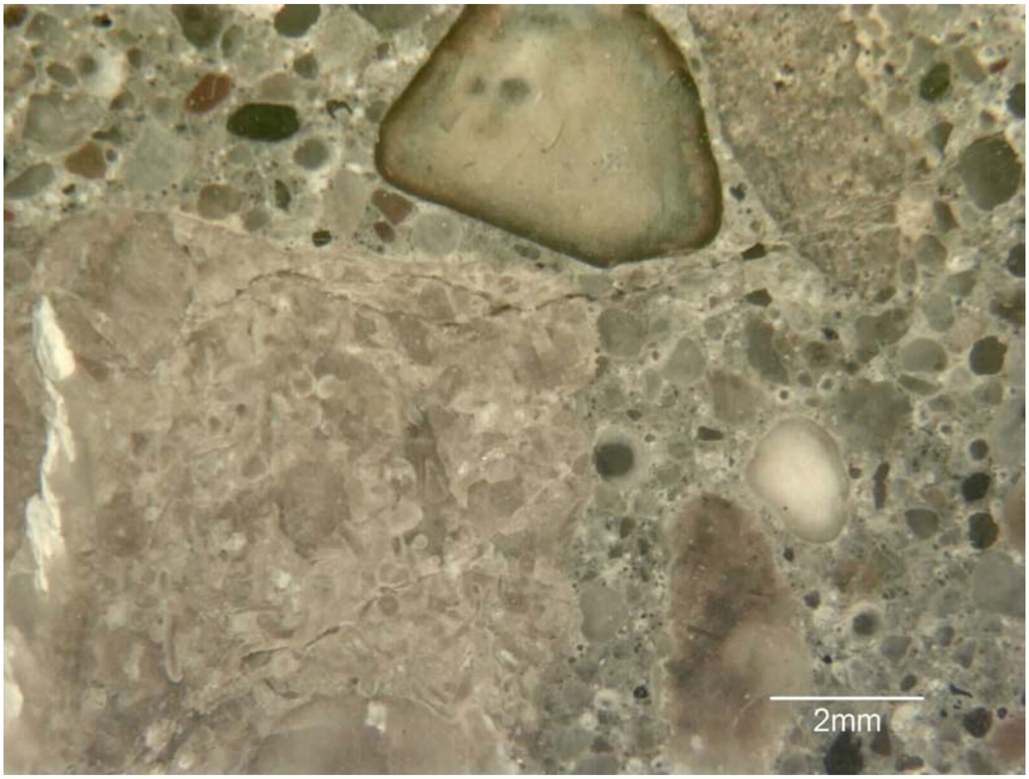


Figure B.3. Core 3: note crack in coarse aggregate particle terminating in paste

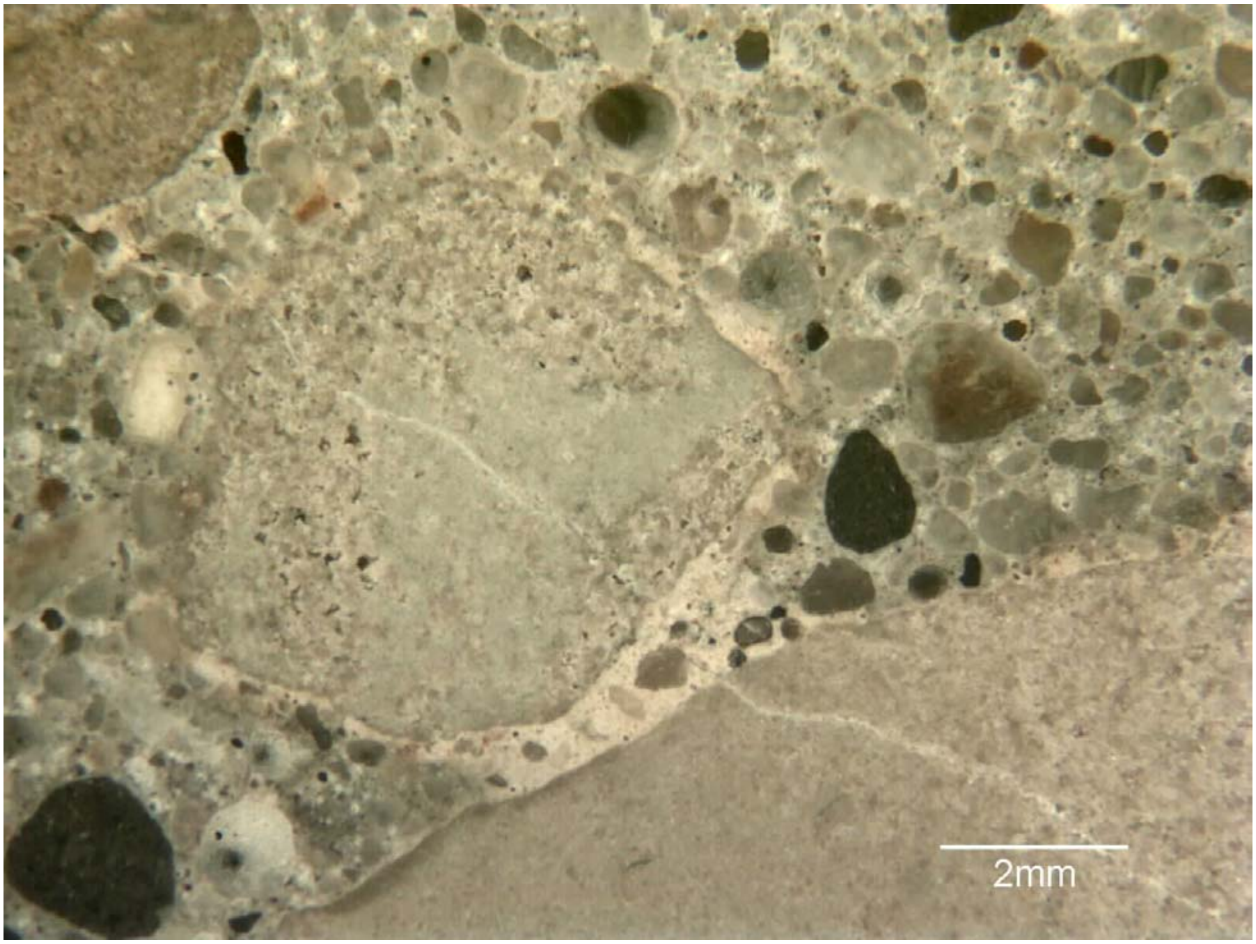


Figure B.4. Core 3: note discolored paste adjacent to coarse aggregate particle. The crack in the aggregate particle near the middle of the image suggests the presence of some ASR.

APPENDIX C: SUMMARY AND GENERAL OBSERVATIONS OF THE CORES EXTRACTED FROM P 73

Table C.1. Summary of Cores from P 73

	Core 1	Core 2
Location	Mid-panel	Joint
Length (inches)	6.9	6.75

County road P73 (just north of US 20) in Webster County is badly map-cracked. Two cores were taken by Iowa DOT personal for petrographic examination. Figures C.1 and C.2 show polished cross sections of the cores. Figures C.3 through C.9 are photomicrographs showing details of fractures in both the aggregate and the paste. Several general observations can be made about the aggregate particles and fractures seen in these two cores. (1) The coarse aggregate is clastic carbonate limestone containing fossil fragments, peloids, and oolites. This is consistent with aggregate from the Gilmore City Formation (probably Fort Dodge Mine). (2) The fractures tended to be at the coarse aggregate/paste boundary (e.g., Figures C.3, C.7, C.8, and C.9). Some of these fractures may have occurred while the paste was still plastic. (3) Some of the fractures extent from the aggregate/paste interface into the paste (e.g., Figures C.4, C.7, C.8, and C.9). (4) Fractures in the aggregate tended to terminate at the aggregate paste interface and not propagate into the paste (e.g., Figures C.6, C.8, and C.9). These observations are consistent with the premature distress being related to construction problems and not aggregate or materials.

Figure C.1. Polished Cross Section of Core 1 from County Road P 73, Webster County

Figure C.2. Polished Cross Section of Core 2 from County Road P 73, Webster County

Figure C3. Optical photomicrograph from the polished section from core 2 showing an oolitic and fossiliferous coarse aggregate particle. Fractures are seen at the aggregate/paste boundary.

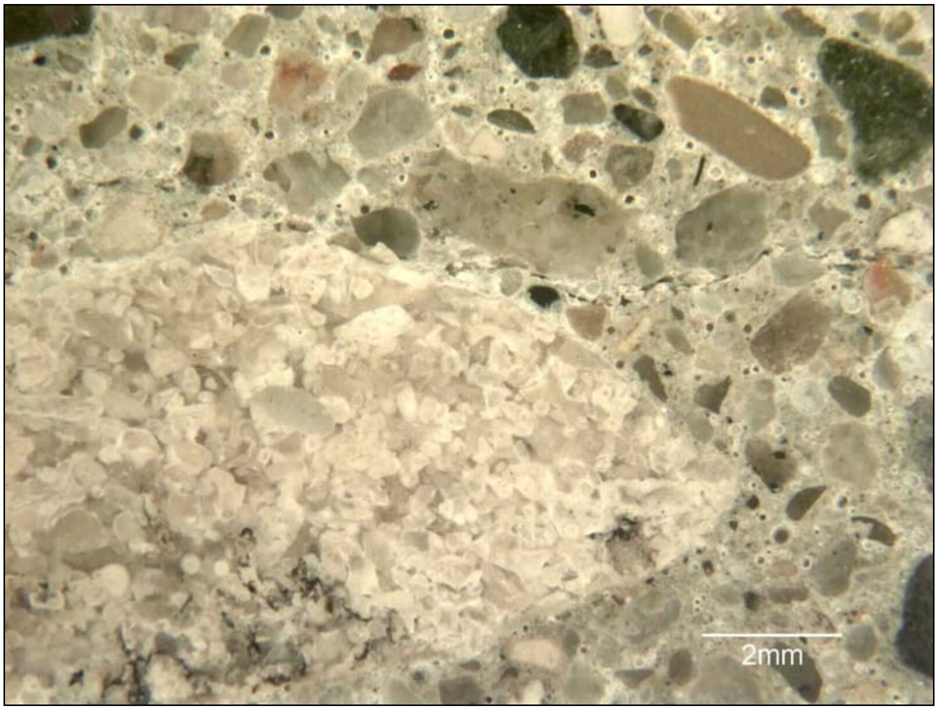


Figure C.4. Optical photomicrograph from core 2 showing another oolitic/peloid coarse aggregate particle. In this case, the fracture extends from the aggregate/paste boundary into the cement paste (core 2).

Figure C.5. Optical photomicrograph showing the clastic components found in the coarse aggregate particles (core 2).

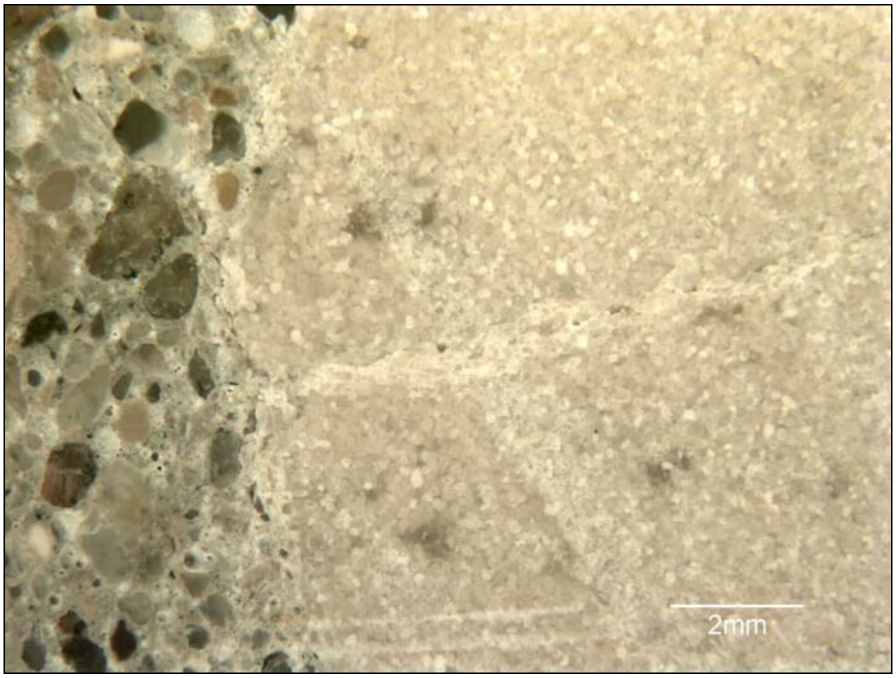


Figure C.6. Example of a fracture in a coarse aggregate particle, which terminates at the paste boundary (core 2). Features such as this indicate the pavement failure was not due to aggregate problems.

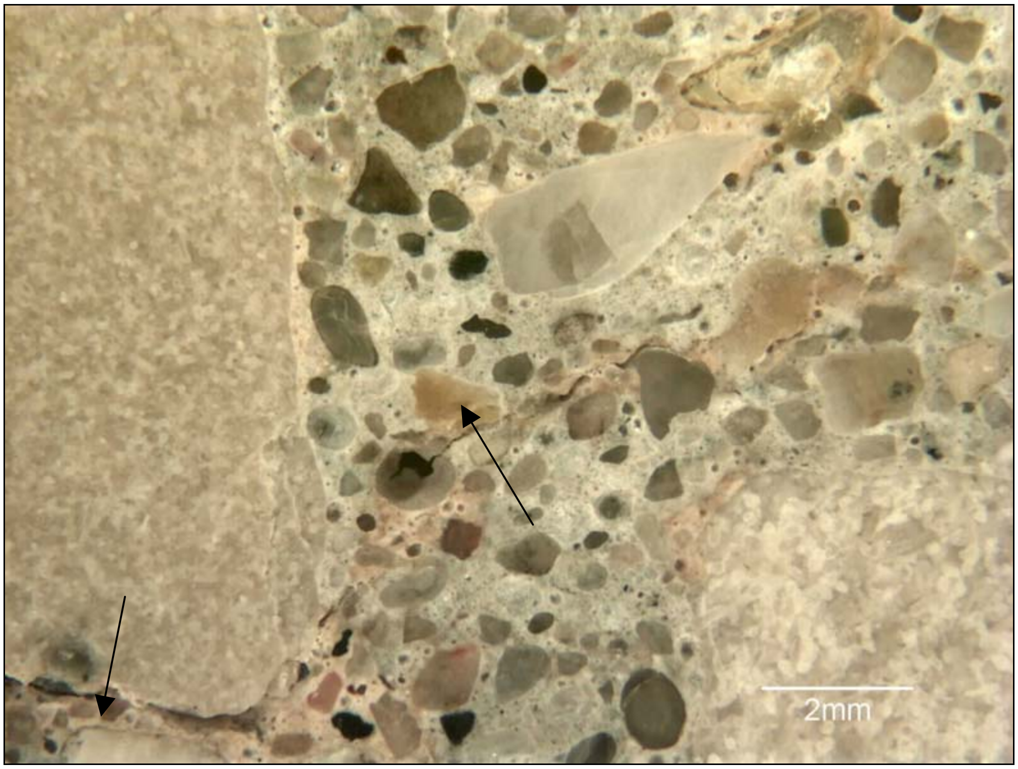


Figure C.7. Example of a fracture along the aggregate/paste boundary extending into the cement paste (core 1).

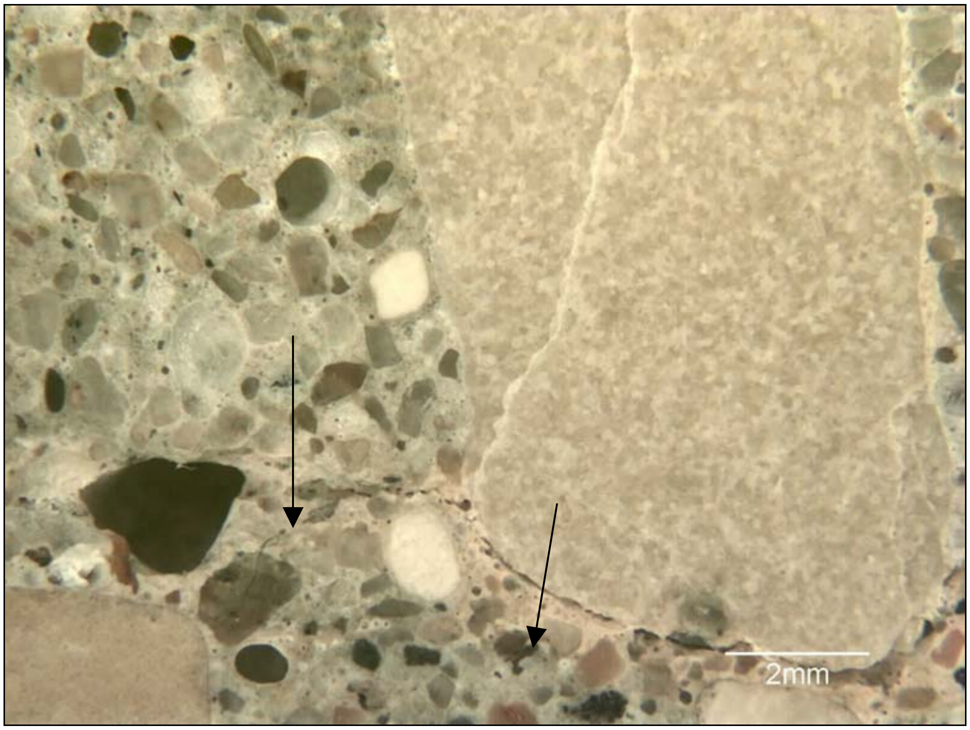


Figure C.8. The fracture along the aggregate paste boundary extends into the paste while the fracture in the aggregate terminates at the paste (core 1).

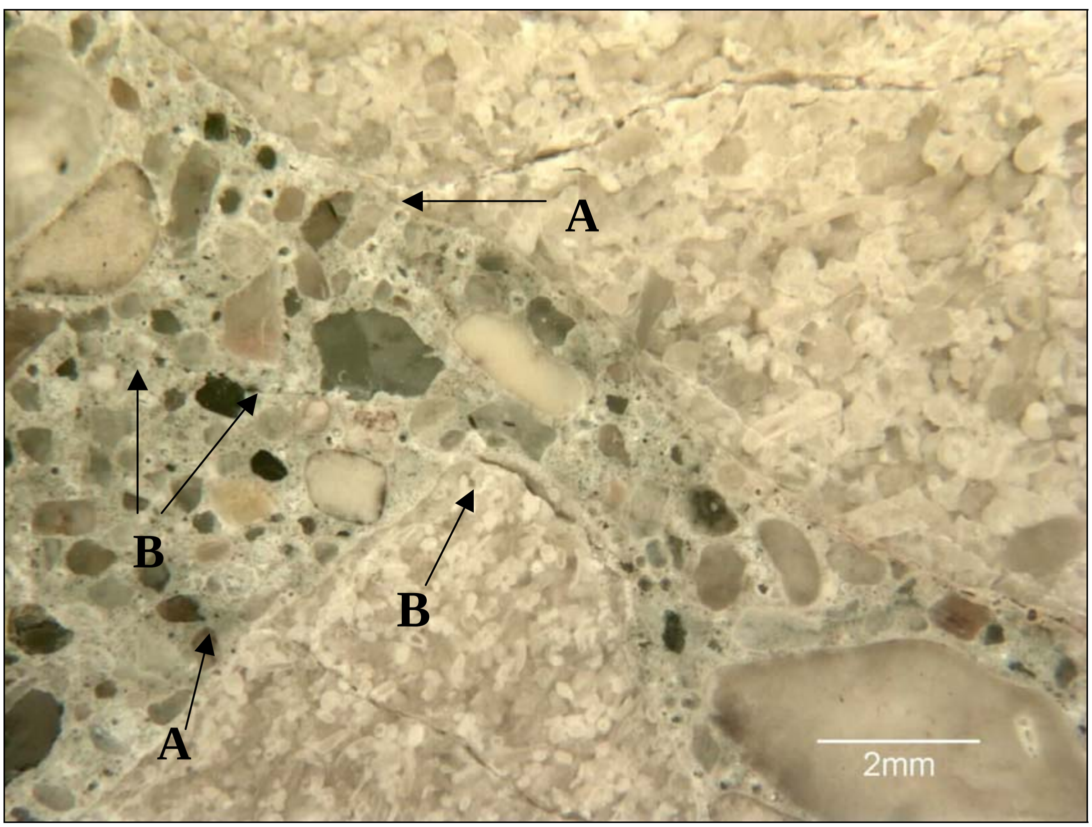


Figure C.9. Fractures in the aggregate (A) terminate at the edge of the particle while fractures at the aggregate/paste boundary extend into the paste.

APPENDIX D: SUMMARY AND GENERAL OBSERVATIONS FROM HWY 175

Highway: Hwy 175/Hamilton WB lane, MP 156.2, sta 143
Mix Details: A-3-F
Coarse Aggregate: Moberly Mine (Alden)
Fine Aggregate: Hallet sand
Cement: Penn Dixie
Fly Ash: Port Neal 3, 17% by mass

Observations: Visual inspection and light microscopy

Core Location & Details	Aggregates	Voids	Cracks	Comments
Trans. Joint Section A	Limestone sound Sand contains shale	Some distorted entrapped voids	Shale, two coarse aggregate particles cracked near wear surface	Slight segregation noted Core length = 8.5”
Trans. Joint Section B	Limestone sound Sand contains shale	Entrapped air voids <5mm diameter	Shale, one coarse aggregate particle cracked	
Trans. Joint Section C	Limestone sound Sand contains shale		Shale, one coarse aggregate particle cracked	
Mid-panel Section A	Limestone sound Sand contains shale	Entrapped air voids <5mm diameter	None, except for cracked shale	One cracked chert particle noted Core length = 8.0
Mid-panel Section B	Limestone sound Sand contains shale	Entrapped air voids <12mm diameter	None, except for cracked shale	

Observations: Scanning electron microscopy

Location & Details	Air Content, % (mortar basis)	Area Weighted Mean Diameter. (microns)	Comments
Trans. Joint Section A	7.0	235	
Trans. Joint Section B	8.1	231	
Trans. Joint Section C	8.8	300	
Mid-panel Section A	7.6	218	
Mid-panel Section B	6.4	204	

Highway: Hwy 175/Hamilton EB lane, MP 157.5, sta 191
Mix Details: A-3-C
Coarse Aggregate: Moberly Mine (Alden)
Fine Aggregate: Hallet sand
Cement: Penn Dixie
Fly Ash: Council Bluffs, 15%

Observations: Visual inspection and light microscopy

Core Location & Details	Aggregates	Voids	Cracks	Comments
Trans. Joint Section A	Limestone sound Sand contains shale	Some distorted entrapped voids	None, except for cracked shale	No surface cracks Core length = 8.5”
Trans. Joint Section B	Limestone sound Sand contains shale		None, except for cracked shale	
Mid-panel Section A	Limestone sound Sand contains shale		None, except for cracked shale	Core length = 8.25”
Mid-panel Section B	Limestone sound Sand contains shale		None, except for cracked shale	

Observations: Scanning electron microscopy

Location & Details	Air Content, % (mortar basis)	Area Weighted Mean Diameter. (microns)	Comments
Trans. Joint Section A	8.1	266	Some voids lined with ettringite, none filled.
Trans. Joint Section B	5.0	221	
Mid-panel Section A	6.8	223	
Mid-panel Section B	5.8	244	

Highway: Hwy 175/Hamilton EB lane, MP 158.3, sta 244
Mix Details: A-3
Coarse Aggregate: Moberly Mine (Alden)
Fine Aggregate: Hallet sand
Cement: Penn Dixie
Fly Ash: None

Observations: Visual inspection and light microscopy

Core Location & Details	Aggregates	Voids	Cracks	Comments
Trans. Joint Section A	Limestone sound Sand contains shale	Some voids lined with white material	None, except for cracked shale	No surface cracks Core length = 8.25”
Trans. Joint Section B	Limestone sound Sand contains shale		None, except for cracked shale	
Mid-panel Section A	Limestone sound Sand contains shale		None, except for cracked shale	Core length = 8.0”
Mid-panel Section B	Limestone sound Sand contains shale		None, except for cracked shale	

Observations: Scanning electron microscopy

Location & Details	Air Content, % (mortar basis)	Area Weighted Mean Diameter. (microns)	Comments
Trans. Joint Section A	10.5	241	Some voids lined with ettringite, none filled.
Trans. Joint Section B	8.8	259	
Mid-panel Section A	8.1	190	
Mid-panel Section B	8.2	227	