Non-destructive and destructive bridge deck condition assessment

Aleksandra V. Varnavina

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NON-DESTRUCTIVE AND DESTRUCTIVE BRIDGE DECK CONDITION

ASSESSMENT

by

ALEKSANDRA VYACHESLAVOVNA VARNAVINA

A DISSERTATION

Presented to the Faculty of the Graduate School of the
MISSOURI UNIVERSITY OF SCIENCE AND TECHNOLOGY
In Partial Fulfillment of the Requirements for the Degree

DOCTOR OF PHILOSOPHY

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Approved by
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J. David Rogers
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Paper 3, pages 52-76, is entitled “Integrated Approach in Assessing Bridge Deck Condition”, and is prepared in the style used by the *Automation in Construction*, as submitted on October 21, 2015.
ABSTRACT

The dissertation is composed of three papers, which cover the lack of information on the specific aspects of non-destructive and destructive bridge deck assessment.

In the first paper, appropriate data acquisition and processing parameters for concrete bridge deck condition assessment using ground-coupled ground penetrating radar are developed. The use of proposed parameters helps to significantly reduce acquisition and processing time, while providing engineers with reliable and detailed information on the condition of bridge deck.

In the second paper, a novel approach to develop relationship between GPR data and concrete removal depth measurements collected after hydrodemolition is proposed. A linear relationship between the two is assumed, justified and corrected. Two case studies are used to verify the proposed approach.

In the third paper, an integrated approach in assessing bridge deck condition is introduced. Four techniques – visual inspection, GPR, USW, and core control – were used to perform a bridge deck assessment. LiDAR measurement of concrete depth removal collected after hydrodemolition were used as ground truth. The advantages and disadvantages of each method are discussed. Qualitative and quantitative comparisons of data collected using non-destructive and destructive techniques are performed.
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1. INTRODUCTION

Bridges are important part of the transportation system, providing roads to cross over most obstacles. Due to harsh environmental conditions and traffic loads, bridge decks tend to deteriorate over time. Bridge deck deterioration is a significant problem that leads to serviceability problems and even failure. To prevent failure and prevent significant damage, proper assessment must be done periodically so that potential problems are addressed in a timely manner.

Non-destructive and destructive methods are used to monitor the health of the bridge decks. Non-destructive methods include Ground Penetrating Radar (GPR), visual inspection, Impact Echo (IE), Ultrasonic Wave (USW), chain drag, infrared thermography (IR), Half-cell potential (HCP), etc. Destructive methods include core control and chloride ion concentration measurements.

As a part of this study, eleven concrete bridge decks in Missouri, USA, were surveyed using both non-destructive (GPR, visual inspection and USW) and destructive (core control) techniques. Three of the concrete bridge decks underwent rehabilitation which included milling and hydrodemolition. Hydrodemolition uses high pressure water jets to remove deteriorated concrete from the top surface of bridge decks. After the hydrodemolition, LiDAR technology was used to measure the thickness of concrete removed.

The goal of this research is to develop a quality improvement for bridge deck assessment using non-destructive and destructive evaluation methods. In this dissertation data acquisition and processing parameters for concrete bridge deck assessment using ground-coupled GPR are presented. In addition to the acquisition and processing parameters, an approach to predict concrete repair quantities based on GPR reflection amplitudes is presented. Another contribution of the work is the introduction of integrated approach in assessing bridge deck condition. The approach allows for identification various deterioration states.
I. Data acquisition and processing parameters for concrete bridge deck condition assessment using ground-coupled ground penetrating radar: Some considerations

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ABSTRACT

Ground penetrating radar (GPR) is a non-destructive geophysical technique that is widely used to determine the relative condition of reinforced concrete. This paper presents case studies from Missouri, USA, where a ground-coupled GPR system was used to assess the condition of eleven concrete bridge decks. The main goal of this paper is to develop appropriate acquisition and processing parameters in order to conduct rapid, efficient, and cost effective assessment of bridge decks. To accomplish this goal, the GPR data sets were collected with slightly different acquisition parameters and processed using different parameters. The quality of the results and the time required for each bridge deck survey are analyzed. Additionally, several experimental data sets were collected across a 12th concrete bridge deck to examine the influence of weather conditions on reflection amplitude values, since amplitude analysis is used in this study. Based on the authors' experience and findings, appropriate GPR acquisition and processing parameters are suggested and described for use of the ground-coupled GPR method for bridge deck assessment.
1. Introduction

Bridges are a significant part of the transportation system, allowing roads to cross over most obstacles. Due to harsh conditions such as exposure to deicing salts, temperature fluctuations, and heavy traffic, bridge decks tend to deteriorate over time. Corrosion of the internal reinforcing steel is a major cause of concrete bridge deck deterioration. Corrosion by products cause the steel to expand and crack the surrounding concrete, allowing for increased deterioration rates (Belli et al., 2013). To prevent and delay significant damage, bridge deck monitoring is essential. Nondestructive techniques, in particular, can be used to identify deterioration at early stages, and the results can help guide decision makers in determining the need for bridge deck rehabilitation or even replacement.

Ground penetrating radar (GPR) has recently been determined to be an effective and efficient technology for bridge deck inspection (Gehrig et al., 2004; Barnes and Trotter, 2004; Tarussov et al., 2013). GPR is a nondestructive tool that uses electromagnetic (EM) signals to penetrate into the medium and measure amplitude and two-way travel time of reflections from the boundary of materials with different electric properties (Shin and Grivas, 2003). GPR is an effective technique for the evaluation of reinforced concrete because of the significant dielectric contrast between concrete and steel. Two types of GPR have been used in bridge deck evaluation: air-launched GPR and ground-coupled GPR. An air-launched GPR antenna is useful to acquire data at higher speed with lower resolution measurements, while the use of a ground-coupled GPR antenna provides higher resolution but lower speeds of data acquisition. Since lane closures are generally required during bridge deck surveys with ground-coupled GPR, such survey needs to be completed rapidly while providing good quality data sets for further processing and interpretation.

As part of this study, eleven concrete bridge decks in Missouri, USA, were surveyed using ground-coupled GPR to assess the condition of the bridge decks. The details of each bridge are discussed in detail elsewhere (Sneed et al., 2014). The data sets from the eleven investigations were obtained using different acquisition parameters and then processed using slightly different processing parameters. The objective of this paper
is to design, develop, and validate appropriate acquisition and processing parameters for concrete bridge deck GPR surveys on the basis of the eleven bridges investigated in this study and supplemented with additional test surveys conducted on a 12th concrete bridge deck.

2. Acquisition parameters

Prior to a GPR survey, care must be taken to select appropriate acquisition parameters to achieve an appropriate balance between cost and data quality. The main acquisition parameters include antenna frequency, number of scans per unit of distance, dielectric constant, range, number of samples per scan, transmit rate, antenna filters, gain, and traverse spacing, all of which are described in the paragraphs that follow. Important aspects in planning and preparation, as well as conditions during data collection, are also discussed in this section.

2.1. Antenna frequency

Investigation of a concrete bridge deck using ground-coupled GPR is frequently performed with the use of one or more high-frequency antennas (greater than 900MHz) to provide an optimum balance between depth and resolution of imaging (Gehrig et al., 2004).

In this study, the bridge deck investigations were performed using a GSSI SIR System-3000 unit coupled with a 1.5 GHz antenna and mounted on a compact hand-pushed cart. Based on the authors' past experience, this GPR system has proved adequate for shallow, high resolution investigations, as it provides high quality data and is easy to operate. GSSI states that a 1500 MHz antenna can image to a depth of 18 in.; a 900MHz antenna can image to a depth of 36 in. (GSSI, 2006). In most investigations, the objective is to image the uppermost layer of reinforcing steel. This was also the objective of the surveys conducted in this study.
2.2. *Number of scans per unit of distance*

The number of scans per unit of distance is a function of EM pulse repetition and acquisition speed. This parameter affects both lateral resolution and acquisition speed.

In an attempt to optimize this parameter for the equipment utilized in the field investigations in this study, a survey was carried out to determine the time required to acquire ground-coupled GPR data that allows clear imaging of individual pieces of rebar for a different number of scans per unit distance. Data were acquired along the same 20 ft. long traverse of a concrete slab using a high frequency (1.5 GHz) GPR antenna, and the resulting graph is presented in Fig. 1 and Fig. 2 shows the GPR data collected at the same location using a different number of scans per distance.

For the same traverse, the influence of numbers of scans per distance on amplitude values was also studied and is shown in Fig. 3. A very minimal effect on amplitudes is observed, which is likely caused by slightly different rebar peak locations as expected, as the amplitudes are picked manually.

![Fig. 1. Number of scans per foot vs. acquisition time recorded per 20 ft. of distance.](image-url)
Fig. 2. GPR scans collected at the same location using a different number of scans per unit of distance: a) 12 scans/ft., b) 24 scans/ft., c) 48 scans/ft., d) 72 scans/ft., e) 96 scans/ft., f) 120 scans/ft., g) 144 scans/ft.

Fig. 3. Reflection amplitude values collected at the same location using a different number of scans per unit of distance.

As seen in Fig. 1, the parameter of 12 scans/ft. allows for collecting data more rapidly. However, using such coarse scan spacing may limit data visibility in the field and cause inaccurate manual adjustments of peaks when processing (Fig. 2a). Conversely, 24 scans/ft. is found to be sufficient to image a single piece of rebar for detailed amplitude analysis and can be used to significantly increase data acquisition speed and obtain good
quality data for further amplitude analysis. However it should be kept in mind that if other field estimates are required (e.g., selecting a site for coring, locating an individual steel bar, imaging of the lower layer of transverse steel), a denser scan spacing is recommended to improve visibility of GPR scans as they are being collected. Clearly, if identifying anomalies in the field is necessary, more care must be taken to investigate those areas.

2.3. Weather conditions & dielectric constant

Weather conditions should also be taken into consideration and documented. Changing weather conditions can cause variations in the moisture content in the bridge deck, and as a consequence alter the dielectric constant of the medium investigated. Small cracks and fractures in concrete tend to hold water increasing both the dielectric constant and conductivity of the material (Tarussov et al., 2013). However, the conductivity of concrete may not be uniform throughout the entire deck due to varying moisture and chloride content. Moisture and chloride content decrease the propagation velocity and reflection amplitude. Fundamentally, propagation velocity \( v \) is a function of the dielectric constant \( \varepsilon \) \( (v = c/\sqrt{\varepsilon}) \), where \( c \) is the speed of light), and EM velocity decreases with increasing dielectric constant. Similarly, signal attenuation might affect reflection amplitude as the signal penetrates through conductive concrete, weakens, and strikes reinforcing steel with less energy (Barnes et al., 2008).

To investigate the influence of weather conditions on the reflection amplitude values, a study was carried out in which reflection amplitude values were measured along a given traverse on a solid reinforced concrete bridge deck (Fig. 4) during different weather conditions. Measurements were carried out using the same GPR antenna with the same acquisition settings over a time period of 6 months (from December 2012 until May 2013). Fig. 5 shows the reflection amplitude results from four different scans corresponding to the following: 1) December 5, 2012, 0.98 in. of rain reported within 35 h prior to the investigation, temperature range of 33–57 °F; 2) February 19, 2013, no precipitation within 24 h prior to the investigation, temperature range of 24–35 °F; 3) May 19, 2013, no precipitation within 24 h prior to the investigation, temperature range of 68–87 °F; and 4) May 20, 2013, 0.60 in. of rain reported within 10 h prior to the
investigation, temperature range of 60–75 °F. Results from scans 3 and 4, which were acquired within 24 h of each other, clearly illustrate a difference in reflection amplitudes at each location along the traverse. Since it is reasonable to assume that the bridge deck did not deteriorate significantly within a single day, the differences in results from scans 3 and 4 can be attributed to differences in weather. Comparing the reflection amplitudes from scans 1-4, it can be observed that the values from scan 4 are larger (less negative) than those of scans 2 and 3. Under the same conditions, these results would suggest that the bridge deck actually improved with time. However, this cannot be the case since no intervention was carried out on the bridge deck within this time period. Therefore, the weather conditions to which the bridge deck was exposed can be assumed to have influenced the results. Additional study is currently underway by the authors to further study this issue. However, these results illustrate that in practice, it is important to complete a GPR survey within one day with no significant weather changes so that the range of reflection amplitude values is consistent. The test data also indicate that the absolute value of the reflection amplitudes is a less critical consideration than relative differences in reflection amplitude.

Fig. 4. General view of a bridge where different measurements were acquired in different weather conditions.

The dielectric constant should be set using core control for calibration purposes. If this is not possible, bridge plans (rebar embedment depth or deck thickness) should be
used for calibration purposes. If this is not possible, the authors recommend using a dielectric permittivity of 6 for good quality concrete and 8 for deteriorated concrete. Irrespective of the dielectric permittivity employed in the field, the user should ensure that the upper layer of rebar is effectively imaged (given the range employed).

Fig. 5. Reflection amplitude mapped along the same distance at different weather conditions. Dashed line shows smoothed amplitude data. 1) December 5, 2012, 2) February 19, 2013, 3) May 19, 2013, 4) May 20, 2013.

2.4. Range

The range corresponds to the two-way travel time window in nanoseconds (ns). Setting a longer time range allows energy to penetrate deeper (GSSI, 2006). To set the range appropriately, the target depth, GPR antenna frequency, and the number of samples per scans should be taken into consideration. The user also needs to ensure that the reflected wavelet is sampled with sufficient density to ensure that the maximum reflection amplitude is digitally recorded.

In general, a time range of 9–12 ns is needed for shallow concrete evaluations, such as those of a concrete bridge deck (GSSI, 2006). Based on the authors' experience, a lower range (9 ns) is to be used for decks in good condition, and a higher time range (12
ns) is required for decks in poor condition after a sufficient amount of precipitation. As an example, Table 1 shows the difference in apparent depth estimated using different values of dielectric permittivity and range associated with good and poor conditions. Note that the range is set a little higher than necessary to pass energy deeper down and allow reflections from deeper layers to be recorded. It is also necessary as concrete is not homogenous, which could cause unforeseen velocity changes. Additionally, a greater range may be necessary if data are to be migrated. The authors recommend acquiring test data to ensure that the upper layer of rebar is imaged.

Table 1
Apparent depth estimates calculated for given values of dielectric permittivity and range (ns).

<table>
<thead>
<tr>
<th>Range, ns</th>
<th>Dielectric permittivity</th>
<th>Apparent depth, in</th>
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<tr>
<td>9</td>
<td>6</td>
<td>21.68</td>
</tr>
<tr>
<td>12</td>
<td>6</td>
<td>28.90</td>
</tr>
<tr>
<td>9</td>
<td>8</td>
<td>18.77</td>
</tr>
<tr>
<td>12</td>
<td>8</td>
<td>25.03</td>
</tr>
</tbody>
</table>

2.5. *Number of samples per scan*

The number of samples per scan can affect the vertical resolution and acquisition speed. Additionally, a greater number of samples per scan requires more computer memory and reduces data acquisition speeds (GSSI, 2006).

To select a number of samples per scan, the time window (range) should be taken into consideration. More specifically, if the range is too great and the number of samples is too small, aliasing can occur. Even if aliasing does not occur, the user should ensure that the reflected wavelet is sampled with sufficient density to ensure that the maximum reflection amplitude is digitally recorded. To investigate the effect of the number of samples per scan on reflection amplitudes, two GPR traverses were collected with the range of 9 and 15 ns and using 2048 samples/scan and then re-sampled into 1024, 512, 256 and 128 samples per scan (Fig. 6). The amplitude values are measured and compared
According to the scans with the range of 9 ns (Fig. 7a), there are no significant variations in amplitude values. The same analysis was performed for the scans collected with the range of 15 ns and demonstrates noticeably larger amplitude variations (Fig. 7b). Clearly, the setting of 128 samples per scan would not be appropriate with a longer trace length, such as those of 15 ns. This would lead to the conclusion that 256 samples per scan is the minimal number that can be used for concrete evaluations performed with the range up to 15 ns.

Fig. 6. GPR scans collected over a bridge deck using range of 15 ns and different number of samples per scan: a) 128 samples/scan, b) 256 samples/scan, c) 512 samples/scan, d) 1024 samples/scan, e) 2048 samples/scan.

Fig. 7. Reflection amplitude values collected with a different number of scans per unit of distance and range of 9 ns (a) and 15 ns (b).
2.6. Transmit rate

The transmit rate corresponds to the speed of data acquisition. Higher transmit rates correspond to faster data collection ability (GSSI, 2006).

An appropriate transmit rate should be chosen in accordance with the manufacturer's recommendations for the GPR model. In this study a transmit rate of 100 kHz was used as recommended for 1.5 GHz antenna. According to the manufacturer, a transmit rate of 100 kHz is the rate at which the 1.5 GHz antenna was tested and rated (GSSI, 2006).

2.7. Antenna filters

It is essential to use filters during data acquisition so that coherent reflections are more visible and can be interpreted as being collected.

Certain antenna filters may be recommended by the GPR system manufacturer to smooth noise and remove interference if it occurs. Although the filters are typically set automatically by the GPR system, manual adjustment is possible to improve visual quality of the data (GSSI, 2006).

In this study, Infinite Impulse Response (IIR) filters (low-pass - 3000 MHz, high-pass - 250 MHz) were used to acquire high quality GPR data. The filters were set by default and determined the range of frequencies that the antenna can receive.

2.8. Gain

It is critically important to adjust the gain. The gain must be properly adjusted so that no part of the signal is clipped (over-gained) (Fig. 8). Some GPR systems are specifically designed to automatically set the gain within acceptable parameters during the initialization period (GSSI, 2006). Normally, bridge deck data are acquired using a one point gain. During the investigations conducted in this study, data were acquired from one bridge using a three point gain to enhance the amplitude of the reflection from the lower mat of reinforcing steel. However, the first two gain points were located above and below the upper layer of reinforcing steel to ensure that all upper layer rebar amplitudes were relative. It is also possible to remove the applied gain during data processing.
Based on the authors' experience, a negative number of decibels is used as a single gain point with the intent to weaken a signal by a certain amount and avoid signal clipping.

![Fig. 8. Gain adjusted properly (right) and improperly which caused signal clipping (left).](image)

2.9. Traverse spacing

Traverse spacing must be chosen depending upon the objective of investigation. In general, coarser traverse spacing requires less survey time but decreases spatial resolution of mapping, which could lead to inaccurate results, especially if deterioration quantities are to be estimated. If the deck is the subject of a detailed survey, a denser spacing is required which increases spatial resolution of mapping.

As an example, reflection amplitude maps for one of the bridge decks investigated in this study are plotted in Fig. 9. Data were collected along traverses spaced at 1 ft. The reflection amplitude map plotted using every traverse (1 ft. spacing) is assumed to be the most detailed to detect areas of deterioration and shown in Fig. 9a. In order to optimize the GPR parallel traverse spacing, the reflection amplitude map was re-plotted and compared using every 2nd, 3rd, 4th, 5, and 6th GPR traverse in Fig. 9b – f, respectively. Fig. 10 shows the distribution of amplitude values for the maps generated in Fig. 9 using GPR traverse spacings of 1, 2, 3, 4, 5 and 6 ft. accordingly. The maps generated with 2 ft. and 3 ft. traverse spacings are approximately equal to the original map with 1 ft. spacing in terms of amplitude distribution (Fig. 10), and there is a more noticeable difference in
the amplitude distribution for the mappings generated with 4 ft., 5 ft., and 6 ft. GPR traverse spacings (Fig. 10).

**Fig. 9.** Reflection amplitude mappings generated with 1ft. (a), 2 ft. (b), 3 ft. (c), 4 ft. (d), 5 ft. (e), 6 ft. (f) GPR traverse spacing.

**Fig. 10.** Bar graph showing percentage of distribution for given values of amplitudes with various traverse spacing.

Fig. 11 shows a cumulative distribution of reflection amplitudes for the maps generated in Fig. 9 using GPR traverse spacings of 1, 2, 3, 4, 5 and 6 ft. Although this
paper does not attempt to define threshold values associated with certain amplitude ranges, the cumulative graph (Fig. 11) shows that traverse spacings of 1 to 3 ft. have similar percent areas of distribution for different amplitude ranges. On the other hand, traverse spacings of 4 to 6 ft. tend to underestimate the percent area associated with high amplitudes and low amplitudes (relative to the percent area determined by the 1 ft traverse spacing).

![Cumulative bar graph showing percentage of distribution for given values of amplitudes with various traverse spacing.](image)

**Fig. 11.** Cumulative bar graph showing percentage of distribution for given values of amplitudes with various traverse spacing.

In cases where zones of deterioration are long and narrow and parallel to the GPR traverse (along the curb or center of roadway, for example), a coarse traverse spacing is inappropriate for detailed survey as it may not identify such zones of degradation. For example, reflection amplitude maps of a bridge deck with a long and narrow zone of deterioration along the center of the deck are shown in Fig. 12. The maps were generated in Fig. 12 using GPR traverse spacings of 1, 2, 3, 4, 5 and 6 ft. The map generated with 2 ft. traverse spacing is nearly identical to the map created with 1 ft. traverse spacing. The maps generated with 3, 4 and 5 ft., however, show a slight shift of the anomaly towards the upper (north-east) edge of the deck.
Fig. 12. Reflection amplitude mappings generated with 1 ft. (a), 2 ft. (b), 3 ft. (c), 4 ft. (d), 5 ft. (e), 6 ft. (f) GPR traverse spacing. A solid line indicates a linear zone of deterioration in the center of the deck.

As seen in Fig. 10f, if the data were acquired with a 6 ft traverse spacing, the zone of degradation along the center of the deck would not have been identified at all.

To summarize, GPR traverse spacing should be selected based upon the actions that are planned for a particular bridge deck. Results in Fig. 9, 10, 11, and 12 show that GPR parallel traverses with a 1 ft to 2 ft spacing would be appropriate for mappings of detailed features (a baseline condition assessment survey, for example). Conversely, a
traverse spacing of 3 ft. to 6 ft. might be used to conduct a reconnaissance survey or a quality assurance.

Additionally, GPR data should be acquired using a zigzag traverse pattern (alternating traverses surveyed in opposite directions) to decrease acquisition time.

2.10. **Planning and preparation**

Planning and preparation are a significant part of the GPR survey. Deck design, including thickness, reinforcement placement, and reinforcement orientation must be considered prior to the survey. Typically, GPR data are acquired perpendicular to the upper layer of reinforcing bars; hence it is advantageous when the upper layer is oriented in the bridge transverse direction (perpendicular to traffic flow), and GPR data can be acquired parallel to traffic flow. The GPR system should be properly calibrated for each type of surface being investigated to obtain accurate distance measurements. After debris is removed from the bridge deck surface, a grid is typically created with chalk or paint on the top surface of the deck to indicate the direction of profiles as shown in Fig. 13. The authors recommend documenting the locations of all cracks, patches, and other visible defects on the deck surface to help interpret the GPR data if needed.

![Fig. 13. GPR field testing on grids.](image)
For the surveys conducted in this study, a minimum of one driving lane (at a time) was closed while the surveys were conducted. The surveys were performed by pushing the cart forward to allow a 2-D GPR image to be generated while walking.

3. Processing parameters

As the goal of this study is to recommend appropriate parameters for rapid and efficient bridge deck assessment, processing should include basic steps only. It should be noted that all GPR data were processed using RADAN 6.5 and RADAN 7, a GPR data software package developed by GSSI.

A zero-time correction must be applied to all GPR scans in order to ensure that zero depth is consistent with the concrete surface. Despite the fact that amplitude analysis was used in this study, the authors assumed that correctly adjusted zero-time was necessary for further processing steps if needed.

Processes such as migration and deconvolution are frequently applied to non-bridge deck data in an effort to increase data resolution both horizontally and vertically (Cardimona, 2002). Migration is a process that essentially collapses hyperbolic diffractions originating from reinforcing bars and moves reflectors to their true subsurface positions (Cardimona, 2002). Deconvolution improves lateral and vertical resolution by increasing the dominant frequency of the wavelets. Typically, migration and deconvolution are not included in bridge deck processing unless there is a strong need for accurate depth estimates analysis or to achieve better resolution.

Once basic processing is completed, reflection amplitude and two way travel time are measured by semi-automatic mapping of rebar reflections. It should be noted that reinforcing bars are represented by hyperbolas where the highest positive peak value is associated with a maximum amplitude value. The primary advantage of semi-automatic mode is that it allows the user to automatically obtain two-way travel time and maximum reflection amplitude information and easily correct mistakenly chosen peak locations as they occur.

An appropriate dielectric permittivity must be selected to transform arrival times to apparent depths understanding that the dielectric permittivity of degraded concrete can
vary significantly across the bridge deck. The best approach is to estimate an appropriate dielectric permittivity based on core control or known deck thicknesses.

Reinforcing steel may not be located at a constant depth (with respect to the top surface of the deck) due to several factors such as construction irregularities or defects, variable surface milling depth, and uneven surface wearing. As a result, the thickness of the concrete cover to the reinforcing steel may vary across the deck. Because the GPR signal attenuates with depth, reflection amplitudes should be normalized to a constant apparent depth using an analytical approach. This approach involves plotting the amplitude versus two-way travel time values to determine a best-fit linear trend and then removing it from the plot by altering amplitude, thus, assigning all reflections to a constant depth (Barnes et al., 2008).

As an example, reflection amplitude is plotted versus two-way travel time for GPR data collected from one of the bridge decks surveyed in Fig. 14a.

![Graph showing reflection amplitude plotted versus two-way travel time before and after depth correction.](image)

**Fig. 14.** Reflection amplitude plotted versus two-way travel time before depth correction (a) and after depth correction (b).

A linear trend line is also plotted in the figure. The steep slope of the trend line in Fig.14a indicates a high influence of depth on reflection amplitude. This could be explained by significant variations in rebar depth within the deck. Also, a significant amount of precipitation (3 in.) was experienced at the site during the three days prior to
the investigation that could cause high signal attenuation with depth. It is assumed that moisture with chlorides penetrated into the concrete, which increased signal attenuation. However, since the penetration of moisture and chlorides may be different at different locations of the deck, the signal attenuation with depth may not be consistent at all locations of the deck. The reflection amplitude is plotted versus two-way travel time after depth correction for the GPR data in Fig. 14b. Original and depth-corrected maps of reflection amplitudes for the same bridge deck are shown in Fig. 15a and b, respectively. It should be noted that the scales used to plot the amplitude maps in Fig. 15 are different.

**Fig. 15.** Reflection amplitude mappings before (top) and after (bottom) amplitude normalization for depth.

After normalization is completed, variations in reflection amplitudes are expected to correspond to deterioration only. This approach is critically important because it removes subtle anomalies associated with inconsistent rebar depth and, therefore, allows for increased accuracy in the analysis of deterioration.
4. Conclusions

GPR is a non-destructive geophysical tool that has been widely accepted by engineering society for many high-resolution applications. GPR data sets that are presented in this paper show that high-frequency (1.5 GHz) ground-coupled GPR antenna can be used for bridge deck investigations providing fast and efficient evaluation of concrete bridge decks. One of the main considerations in using a ground-coupled antenna is that it requires a significant amount of time for data acquisition, and therefore causes traffic disruption. To reduce the time and cost of bridge deck inspections, appropriate data acquisition and processing parameters are examined and offered in this study.

Using the acquisition parameters discussed in this paper, a ground-coupled GPR survey can be accomplished relatively quickly and efficiently. The processing procedure described in this paper can be completed within a few hours, while providing engineers with reliable and detailed information on the condition of the concrete bridge deck.

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References


II. Concrete bridge deck assessment: Relationship between GPR data and concrete removal depth measurements collected after hydrodemolition


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ABSTRACT

A ground-coupled ground penetrating radar (GPR) system was used to assess the condition of two reinforced concrete bridge decks. After each GPR assessment was completed, the bridge deck was rehabilitated using a hydrodemolition process to remove deteriorated concrete from upper surface of the deck. LiDAR technology was used to create maps depicting the deck surface before and after the concrete removal. The objective of this work was to corroborate the GPR condition assessments by comparing the spatial distribution of the GPR and LiDAR mappings. This work illustrates that GPR data have the potential to predict concrete repair estimates.
1. Introduction

Concrete bridge decks degrade over time because of physical stresses and chemical attack. Causes of bridge deck deterioration can include traffic vibrations, freeze-thaw cycles, application of deicing salts, and carbonation. One of the major causes of bridge deck deterioration is corrosion of embedded reinforcing steel, which is usually the result of exposure to moisture and chloride ions. Saline moisture ingresses into a bridge deck typically from the top surface of the deck. This saline moisture penetrates into the concrete to the reinforcing steel, breaking down the passive layer protecting the steel from corrosion. When reinforcing steel corrodes, it expands causing tensile stresses that mechanically weaken and further degrade the encompassing concrete (Fig. 1). Cracking induced within the concrete can progress toward the surface of the deck, providing a path for additional contaminants to penetrate the concrete, or cause delamination along the plane of the reinforcing bars. Bridge deck deterioration, once started, will not stop without intervention. Thus, detection and assessment of bridge deck deterioration is crucial to minimize maintenance and repair costs.

![Fig. 1. Mechanism of reinforcing steel corrosion causing deterioration of concrete over time.](image)

There are several approaches to repair and rehabilitate deteriorated concrete bridge decks. Typically, the most efficient and cost-effective method is chosen depending upon the condition of the structure and estimated extent and thickness of deteriorated concrete. Partial-depth repair is often used if the deterioration is confined mostly to the region of concrete above the upper layer of reinforcing steel (Fig. 2a). Full-depth repair is
often required for cases where concrete deterioration extends beneath the upper layer of reinforcing steel (Fig. 2b). The quantity of each repair type is usually estimated in terms of bridge deck surface area before the repair work begins. Differences in estimated and actual repair quantities can lead to cost overruns and inflated unit prices to account for inaccuracies and probability of losses.

Fig. 2. Concrete slab deterioration caused by corrosion of upper layer (a) and both upper and lower layers (b) of reinforcing steel.

Methods commonly used to estimate concrete bridge deck repair quantities include visual inspection, sounding (i.e., chain drag), and half-cell potential. Visual inspection of the top and bottom deck surfaces, curbs, drains, and other features is an indirect method used to locate indicators of deterioration or distress such as cracking, spalling, rust stains, moisture, or efflorescence. The chain drag method is used to locate delaminations within the concrete as indicated by variations in sound as a chain is dragged over the top surface of the deck [1]. Half-cell potential is used to identify regions of probable corrosion by estimating the electrical corrosion potential of uncoated reinforcing steel embedded within the concrete [2]. Chain drag and half-cell potential methods require removal of the bituminous overlay, if present.

An experienced investigator can usually provide a sufficiently accurate estimation of the quantities of partial-depth and full-depth repairs in terms of bridge deck surface area based on visual inspection alone [3], although the results can be highly variable and depend on the individual inspector [4]. Bridge decks with very little distress are relatively easy to diagnose, whereas it is more challenging to estimate repair quantities accurately for decks with moderate or severe amounts of distress [3]. The chain drag method is commonly used to delineate regions estimated to be in need of repair, although
actual quantities tend to be larger than those estimated on this basis [4]. Several recent studies have examined the use of ground penetrating radar (GPR) to identify regions of bridge deck deterioration [3-6]. GPR reflection amplitude data have also been shown to be capable of indicating the presence of corrosion of reinforcing steel embedded in concrete [7]. A recent study by Barnes and Trottier [3] investigated the use of an air-coupled GPR system in predicting bridge deck repair quantities on a suite of asphalt-covered reinforced concrete bridge decks. The effectiveness was evaluated by comparing GPR-predicted deteriorations to deteriorations detected using chain drag and half-cell potential methods. In their study, the chain drag and half-cell potential data served as ground-truth that was used to form the basis of repair quantity estimation; the actual deterioration and repair quantity were not determined.

In the present study, a ground-coupled GPR system was used to investigate the condition of two reinforced concrete bridge decks. After the GPR investigations were completed, the bridges underwent rehabilitation that included milling of the deck surface followed by hydrodemolition. Detailed survey mappings of the bridge deck surface before the repair initiated and then after hydrodemolition was completed provided a unique opportunity to compare the GPR data with the actual repair data in terms of spatial and quantitative correlations, where the comparison of the pre-rehabilitation and post-hydrodemolition survey data served as ground-truth. This comparison is more accurate than chain-drag or other data acquired using other non-destructive methods, since it indicates the actual amount and location of deteriorated concrete material removed from the deck during the repair. To the authors’ knowledge, such a comparison has not been published in the literature. Another objective of this work was to examine a possible relationship between GPR reflection amplitude and concrete removal depth after hydrodemolition. Such a relationship would be a significant advancement in terms of bridge deck damage location and repair quantity estimations.

2. Background - GPR for bridge deck assessment

Ground penetrating radar (GPR) is a non-destructive geophysical technique that uses electromagnetic (EM) energy to transmit into the subsurface. The transmitted energy
is reflected back from an object or interface that has different dielectric properties than the surrounding material (Fig. 3). The remaining energy then propagates further and gradually diminishes with time. The propagation of the EM signal is highly dependent on the dielectric permittivity and electrical conductivity of the material being tested. The dielectric permittivity controls the speed of the EM signal, and the electrical conductivity determines signal attenuation. The GPR unit measures the amplitude and travel times of EM signal that has been reflected, and are a function of variations in dielectric properties.

GPR is considered to be an effective and efficient tool for assessing bridge deck condition (e.g., [3-6], [8-10]). Air-launched GPR systems are more typically used for rapid or reconnaissance surveys, whereas ground-coupled GPR systems provide for more detailed data analysis and are normally used for detailed investigations, such as those conducted by the authors discussed in this paper [8].

Fig. 3. Diagram illustrating ground penetrating radar survey of a bridge deck.
GPR data acquired on a concrete bridge deck can be analyzed either visually or numerically or both [6]. Visual analysis involves the identification (often in real time) of anomalies identified on essentially continuous 2-D scans and includes two main parameters - variations in reflection travel times and variations in reflection magnitudes. Numerical analysis is typically performed with the use of software that allows picking up reflections and the quantitative measurements of their magnitudes and arrival times. The magnitude and travel time information is transformed (via interpretation) into plan view maps depicting spatial variations in concrete condition. Core control is normally used to constrain and verify the interpretations.

An example of a GPR scan is shown in Fig. 4. As shown, the horizontal scale represents distance along the profile (ft.), and the vertical scale is two-way travel time (ns). Reinforcing bars are represented by hyperbolas where the highest positive peak value is associated with a maximum amplitude value. Evidence of concrete deterioration is noticed as blurred areas with an increase in two-way travel time. Good (or consistent) quality concrete is expected in areas of the scan without variations in apparent depth and weakened reflection amplitudes. Based on the authors’ past experience [11], a deterioration threshold can be determined by visual evaluation of the GPR scans. The visual evaluation involves identifying amplitude ranges for regions with and without evidence of deterioration. Reflection amplitudes in range of 6 - 9 NdB below the maximum value were considered as strong reflections associated with areas with no evidence of deterioration. Weaker reflections that are not in the range of the first 6 - 9 NdB from the maximum amplitude were considered to be associated with deterioration.

The presence of saline moisture in a concrete bridge deck increases the dielectric constant and conductivity of the concrete. Increases in reflection travel time and signal attenuation are often associated with deterioration; however, they can also be associated with conditions that are favorable for the development of deterioration [3]. It should be noted that variations in travel time and signal attenuation can have other causes such as variation in reinforcing bar position (i.e., elevation) or concrete cover thickness, the presence of a different material used in a localized repair region [12], irregularities at the surface of a corroded reinforcing bar, etc. [7]
3. Case study descriptions

3.1. Descriptions of case study bridge decks

Bridge 1, constructed in 1972, is a three-span continuous steel girder system. The bridge deck is a solid cast-in-place concrete slab. The deck is 46 ft. - 10 in. (14.3 m) wide, and the total structure length is 157 ft. (47.8 m). The concrete deck thickness is 7.5 in. (190 mm), with the top layer of reinforcing steel oriented in the transverse direction of the bridge (perpendicular to traffic flow). The top reinforcing steel bars are located at a depth of 1.875 in. (48 mm) (to top of bars) and are spaced 5 in. (127 mm) center-to-center based on the design drawings. Fig. 5 shows a longitudinal cross-section of Bridge Deck 1.

Bridge 2 was constructed in 1966 to provide vehicular traffic over a waterway. The bridge deck is 35 ft. - 4 in. (10.8 m) wide, and the total structure length is 868 ft. (264.6 m). The five-span structure is a continuous steel girder system with a solid cast-in-place reinforced concrete deck; the deck design thickness is 7.5 in. (190 mm). The top layer of reinforcing bars were oriented in the longitudinal direction of the bridge (parallel
to traffic flow) and were spaced 12 in. (305 mm) center-to-center. The transverse (perpendicular to traffic flow) bars directly beneath were spaced 6 in. (152 mm) center-to-center. Fig. 6 shows a transverse cross-section of Bridge Deck 2.

![Fig. 5. Longitudinal cross-section of Bridge Deck 1. (1 in. = 25.4 mm).](image)

3.2. Rehabilitation of case study bridge decks

After the GPR surveys were completed, the two bridge decks were rehabilitated using a procedure that included milling of the surface followed by hydrodemolition. Hydrodemolition is considered to be a cost-effective method for bridge deck remediation,
as it is fast, effective, and minimally impacts the environment [13]. Using hydrodemolition instead of traditional impact type removal methods such as milling or jack hammering is expected to prolong the life of the bridge deck because micro cracking is not induced into the surrounding concrete. Water jets with a constant pressure in the range of 14,000 to 20,000 psi are used to remove deteriorated concrete from the top surface of the bridge deck [14], leaving the sound concrete in place. Deteriorated concrete is typically removed in a single pass; a second pass might be made if needed. The hydrodemolition process also removes corrosion from exposed reinforcing steel and roughens the deck surface to provide adequate adhesion to the new overlay.

For the bridges in this study, the top 0.25 in. (6 mm) of the deck surface was removed using a mill. Milling left behind a rough, grooved surface needed for the hydrodemolition process. After milling was completed, hydrodemolition was used to remove a target minimum of approximately 0.5 in. (13 mm) of concrete from the deck surface as well as any deteriorated concrete beneath. Corroded reinforcing bars were exposed in some locations (Fig. 7). A target minimum of 0.75 in. (19 mm) of concrete was removed from each bridge deck surface by the milling and hydrodemolition processes combined. Fig. 7 and 8 show the surface of Bridge Decks 1 and 2, respectively, after hydrodemolition.

Fig. 7. Bridge deck surface with exposed corroded rebar after removal of deteriorated concrete by hydrodemolition (Bridge Deck 1).
4. Data collection and processing

4.1. GPR survey

The GPR survey of both bridge decks was performed using a ground-coupled GSSI SIR System-3000 unit and a 1.5 GHz antenna mounted on a compact hand-pushed cart (Fig. 9) with the objective of collecting reflection amplitudes from the top layer of transverse reinforcing bars. For Bridge Deck 1, GPR data were acquired in the longitudinal direction along a total of 42 traverses spaced at 1 ft. (305 mm). The acquisition parameters employed were 256 samples/scan, 120 scans/second, and 48 scans/ft. (157 scans/m). The dielectric constant was assumed to be 10.0 [11].

For Bridge Deck 2, the location of the longitudinal bars were identified and marked on the deck surface prior to the GPR survey to acquire data in between them and obtain amplitude information from transverse layer of reinforcing bars. Due to time constraints, GPR data were collected along 12 traverses spaced at 2 ft. (610 mm). Additionally, the shoulders were not investigated, allowing an offset of 3.5 ft. (1.1 m) and 4.5 ft. (1.4 m), respectively. The acquisition parameters were 512 samples/scan, 120/scans/second, and 48 scans/ft. (157 scans/m). The dielectric constant was assumed to be 10.0 [11].
The GPR data were processed using RADAN 6.5, a GPR data software package developed by GSSI [15]. With the use of the Macro command, zero-time correction was performed on the entire data set. Amplitude normalization for variations in concrete cover depth [4] was applied to eliminate undesirable anomalies. For further analyses purposes, the concrete within the bridge decks, and therefore its dielectric constant, was assumed to be uniform.

4.2. Bridge deck surface surveys

In this study, the surface of each bridge deck was surveyed twice using LiDAR (Light Detection and Ranging) with the objective of generating plan view maps depicting the concrete surface. LiDAR is a remote sensing technology that uses laser pulses to determine the range to a target by measuring the time delay between transmission and detection (time of flight) of the reflected signal. This technology is used for a wide range of applications, such as topographic mapping, 3-D surface modelling, and infrastructure studies. This technique is very accurate and fast as it collects large number of points (tens to hundreds of thousands) per second [16].

Fig. 9. GPR data acquisition on Bridge Deck 1.
The first LiDAR survey measurements were made before milling and hydrodemolition (pre-rehabilitation). The second survey measurements were made after the hydrodemolition process was complete (Fig. 10a). Control points, for the purpose of registering the before and after images, were identified on the guard rails of the bridge. The objective of performing the pre-rehabilitation and post-hydrodemolition deck surveys was to determine the spatial variation of the thickness of concrete removed from the bridge deck surface during the rehabilitation process. Comparing and subtracting the two surface maps, the final result was a plan view surface map depicting the elevation difference between the pre-rehabilitation and post-hydrodemolition data that represents the thickness of concrete removed from the top surface of the deck. Based on tests conducted before the actual measurements, this scanning methodology was able to detect variations in elevation on the order of 0.2 in. (5 mm) or better.

Each bridge deck was separated into sections to allow the LiDAR measurements to be made section by section. The resolution of the LiDAR scans ranged from about 0.08 in. (2 mm) at the end of the section closest to the scanner and about 0.4 in. (10 mm) at the end of the section furthest from the scanner. Fig. 10b shows an example image of a section of bridge deck. The LiDAR measurements were then imported into a spreadsheet indicating location and depth of concrete removal for each point.

Fig. 10. LiDAR data acquisition after hydrodemolition (a), LiDAR image collected on bridge deck after hydrodemolition and showing depth difference between pre-rehabilitation and post-hydrodemolition LiDAR data (b). (1 in. = 25.4 mm).
5. Data presentation

5.1. GPR reflection amplitudes and concrete removal depths

Fig. 11 illustrates the mappings of the GPR reflection amplitude from the top layer of transverse rebar and the concrete removal depth (determined from the two LiDAR surveys) for Bridge Deck 1. In this figure, it is visually noticeable that there is a strong correlation between GPR magnitude data (NdB) and concrete removal depth measurements (in.). Areas with lower (more negative) reflection amplitude tend to be located in areas where the survey results indicated a larger thickness of concrete removal. Conversely, areas with higher (less negative) reflection amplitude tend to be located in areas where a shallower depth of material was removed.

Fig. 11. Reflection amplitude mappings (top), LiDAR mappings of concrete removal depth (bottom) of Bridge Deck 1. The two orthogonal dashed lines in each figure indicate locations of cross-sectional profiles. (1 in. = 25.4 mm; 1 ft. = 0.3048 m).

For Bridge Deck 2, the authors analyzed two sections (48x24 ft. and 65x24 ft. size [17x7 m and 20x7 m]) of the bridge deck. Fig. 12 illustrates the GPR reflection magnitude and concrete removal depth mappings. In this plot, it is visually noticeable that there is a considerable correlation between GPR magnitude data (NdB) and concrete
removal depth measurements (in.). Areas with lower (more negative) reflection amplitude tend to be located in areas where the LiDAR map indicated a larger thickness of concrete removal. Conversely, areas with higher (less negative) reflection amplitude tend to be located in areas where a shallower depth of material was removed. It should be noted that correlation between the two maps was expected to be lower than in the case of Bridge Deck 1, since the GPR map was generated with a coarser (2 ft. [610 mm]) traverse spacing, and the LiDAR survey was performed with a dense grid of measurements over the entire deck. In this regard, the authors recommend a denser GPR traverse spacing (1 ft. [0.3 m]) in order to obtain more accurate deterioration estimates.

**Fig. 12.** Reflection amplitude mappings (top), LiDAR mappings of concrete depth removal (bottom) for two sections of Bridge Deck 2. (1 in. = 25.4 mm; 1 ft = 0.3048 m).

### 5.2. Concrete repair quantities

Concrete repair quantities were calculated in terms of surface area for different concrete removal depth ranges based on the survey mapping of concrete removal and are presented in Table 1. The ranges shown in Table 1 are presented in two ways. First, the ranges are defined by key values corresponding to the target minimum removal depth (0.75 in. [19 mm] by milling and hydrodemolition) and the depth to the top of the transverse reinforcing bars to define partial-depth and full-depth repairs for this study. Additionally, ranges are shown in terms of 0.5 in. (12.7 mm) increments to a depth of 3.0
in. (76.2 mm) for additional analysis presented in Section 6. For Bridge Deck 1, it should be noted that the quantities were calculated for the area investigated by GPR only (6396 ft.\(^2\) [594 m\(^2\)]), not for the entire area of bridge deck (7347 ft.\(^2\) [683 m\(^2\)]). For Bridge Deck 2, the quantities were calculated for the two segments analyzed with a total area of 2712 ft.\(^2\) (252 m\(^2\)).

6. Analysis

As mentioned in Section 1, one of the objectives of this paper is to describe a possible relationship between GPR reflection amplitude and concrete removal depth data. As discussed in Section 2, the variations in the GPR reflection amplitudes were assumed to be the result of deterioration. Then, it was assumed that concrete removed during hydrodemolition was deteriorated. Thus, it was assumed that the two data sets are related, however no presumption of direct causality was made.

Using Surfer® [17] the GPR and LiDAR survey mappings were digitized, and then values at corresponding locations were plotted in a scatter plot format to determine statistical regression. The authors assumed a simple linear regression model using the least square method. Fig. 13a shows the scatter plot of approximately 9,000 data points obtained across the surface of Bridge Deck 1.

To justify a linear model the authors considered two approaches. First, a correlation coefficient \( r \) was determined as it measures the strength and direction of a linear relationship. The correlation coefficient \( r = -0.54 \) for the Bridge Deck 1 data in Fig. 13a suggests a moderate negative relationship between the two. In the second approach, cross-sectional plots were generated by digitizing two profiles along and across the deck (Fig. 14). As shown in Fig. 14, an inverse linear relation is observed, as the positive peaks of the concrete removal depth data correspond to the negative peaks of the GPR amplitude data for the majority of the profiles along (Fig. 14a) and across (Fig. 14b) the deck. In other words, when the concrete removal depth data are increased by a constant, the GPR data are decreased by a constant as well. Although the constants are not consistent for the entire profiles, the assumption of a linear relation appears to be a reasonable first attempt.
Table 1
Concrete repair quantities measured from LiDAR survey mapping.

<table>
<thead>
<tr>
<th></th>
<th>Ranges of partial and full depth repairs</th>
<th>Ranges of 0.5 in. (12.7 mm) increments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Depth of concrete removed, in. (mm)</td>
<td>Depth of concrete removed, in. (mm)</td>
</tr>
<tr>
<td></td>
<td>Area, ft $^2$ (m$^2$) Area, %</td>
<td>Area, ft $^2$ (m$^2$) Area, %</td>
</tr>
<tr>
<td>Bridge Deck 1</td>
<td>0.75 and less (19 and less)</td>
<td>0.5 and less (12.7 and less)</td>
</tr>
<tr>
<td></td>
<td>1318 (122) 20.6</td>
<td>300 (28) 4.8</td>
</tr>
<tr>
<td></td>
<td>0.75 – 1.825 (19 – 46)</td>
<td>0.5 – 1.0 (12.7 – 25.4)</td>
</tr>
<tr>
<td></td>
<td>3818 (355) 59.7</td>
<td>2547 (237) 39.8</td>
</tr>
<tr>
<td></td>
<td>1.825 and greater (46 and greater)</td>
<td>1.0 – 1.5 (25.4 – 38.1)</td>
</tr>
<tr>
<td></td>
<td>1260 (117) 19.7</td>
<td>1765 (164) 27.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5 – 2.0 (38.1 – 50.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>764 (71) 11.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.0 – 2.5 (50.8 – 63.5)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>622 (58) 9.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.5 – 3.0 (63.5 – 76.2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>346 (32) 5.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.0 – 4.2 (76.2 – 106.7)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>52 (5) 0.8</td>
</tr>
<tr>
<td>Bridge Deck 2</td>
<td>0.75 and less (19 and less)</td>
<td>0.5 and less (12.7 and less)</td>
</tr>
<tr>
<td></td>
<td>1028 (95) 37.9</td>
<td>486 (45) 17.9</td>
</tr>
<tr>
<td></td>
<td>0.75 – 2.5 (19 – 64)</td>
<td>0.5 – 1.0 (12.7 – 25.4)</td>
</tr>
<tr>
<td></td>
<td>1481 (138) 54.6</td>
<td>1082 (101) 39.9</td>
</tr>
<tr>
<td></td>
<td>2.5 and greater (64 and greater)</td>
<td>1.0 – 1.5 (25.4 – 38.1)</td>
</tr>
<tr>
<td></td>
<td>203 (19) 7.5</td>
<td>438 (41) 16.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5 – 2.0 (38.1 – 50.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>290 (27) 10.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.0 – 2.5 (50.8 – 63.5)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>212 (20) 7.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.5 – 3.0 (63.5 – 76.2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>129 (12) 4.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.0 – 4.8 (76.2 – 121.9)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>75 (7) 2.8</td>
</tr>
</tbody>
</table>


As mentioned in Section 3.2, milling and hydrodemolition combined was designed to remove a target minimum of approximately 0.75 in. (19 mm) of concrete material, so that the authors considered only depths of 0.5 in. (13 mm) and greater when performing further calculations.

**Fig. 13.** Concrete depth removal measurements obtained by LiDAR plotted versus GPR reflection amplitude values (Bridge Deck 1). Linear trend line is chosen (a) and represented with percentage error bars along every 0.5 in. (b). (1 in. = 25.4 mm)

**Fig. 14.** Cross-sectional plots: GPR reflection amplitude (NdB) and LiDAR measurements of concrete removal depth (in.) are plotted along (a) and across (b) the deck to observe a near linear relationship between the two data sets for Bridge Deck 1. (1 in. = 25.4 mm; 1 ft. = 0.3048 m).
Using the simple linear regression model, the equation of the fitted regression line was obtained as shown in Fig. 13a. Then, this fitted regression line was used to compute the fitted (predicted) values of concrete removal depth (variable x) treating GPR amplitude data as fixed. Using the fitted values of concrete removal depth, the distribution of concrete removal depth, predicted using the fitted regression line, was computed in terms of surface area of the bridge deck. The predicted areas of minimum concrete removal, partial-depth repair, and full-depth repair were 36.9, 34.0, and 29.1%, respectively. Comparing these values with the concrete repair quantities measured using LiDAR shown in Table 1, it can be seen that relatively large differences occur. To further examine the distribution of data, the measured and predicted concrete removal depth measurements are compared in Fig. 15a in terms of 0.5 (13 mm) increments. Fig. 15a presents the comparison in terms of cumulative bar graphs showing the percentage of bridge deck surface area for different concrete removal depth ranges: 0.5 – 4.2 in. (13 – 107 mm), 1.0 – 4.2 in. (25 – 107 mm), 1.5 – 4.2 in. (38 – 107 mm), 2.0 – 4.2 in. (51 – 107 mm), 2.5 – 4.2 in. (64 – 107 mm), and 3.0 – 4.2 in. (76 – 107 mm). Relatively large differences between the two bars (up to 23.2%) suggest that the equation of linear regression should be corrected to obtain a better fit.

In an attempt to correct the equation, amplitude error bars (±10%, ±20%, ±30%, ±40%) were added to the original scatter plot in terms of variable y (Fig. 13b). The reflection amplitudes (variable y) were re-calculated using the equation obtained previously, and the errors were added/subtracted, accordingly. The new fitted reflection amplitude values were then used to re-calculate concrete removal depth data (variable x). The original (based on the LiDAR survey results) and re-calculated concrete removal depth data were plotted in the cumulative graph shown in Fig. 16. As shown in Fig. 16, the concrete removal depth data calculated using the initial fitted regression line (green line) underestimates the percent area associated with very shallow removal thicknesses (i.e., less than 1 in. [25 mm]) and overestimates percent area associated with deep removal thicknesses (i.e., below the top layer of reinforcing steel). To address this issue, a closer match should be determined for each category of the cumulative values.

To obtain a closer match, areas of distribution (%) were calculated for each amplitude value (initial and with errors) for each depth category. Then, these percentages
were compared with LiDAR measurements obtained from each depth category. Table 2 was generated using GPR and LiDAR areas of distribution information. GPR percentages closest to the LiDAR percentages were determined for each depth category (indicated in bold red in the table) and are plotted in Fig. 16 for comparison.

From Fig. 16, the magnitude values that showed a best match for each concrete removal depth category are plotted in Fig. 17. A revised linear trend was computed and compared with the initial fitted regression line before correction. The revised fitted regression line is steeper to compensate for the underestimates and overestimates noted above.

Table 2
GPR and LiDAR areas of distribution for each depth category.

<table>
<thead>
<tr>
<th>Concrete Depth Removal, in. (mm)</th>
<th>GPR data (%)</th>
<th>LiDAR data (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-40%</td>
<td>-30%</td>
</tr>
<tr>
<td>0.5 - 4.2 (12.7-106.7)</td>
<td>38.9</td>
<td>46.1</td>
</tr>
<tr>
<td>1.0 - 4.2 (25.4-106.7)</td>
<td>25.0</td>
<td>29.2</td>
</tr>
<tr>
<td>1.5 - 4.2 (38.1-106.7)</td>
<td>15.3</td>
<td>19.8</td>
</tr>
<tr>
<td>2.0 - 4.2 (50.8-106.7)</td>
<td>7.2</td>
<td>11.3</td>
</tr>
<tr>
<td>2.5 - 4.2 (63.5-106.7)</td>
<td>1.9</td>
<td>4.6</td>
</tr>
<tr>
<td>3.0 - 4.2 (76.2-106.7)</td>
<td>0.3</td>
<td>1.1</td>
</tr>
</tbody>
</table>
To validate the improvement of the revised fitted regression line, a comparison of the bar graphs showing the original concrete removal depth measurements (as determined by the LiDAR survey data) and those calculated with the revised equation is shown in Fig. 15b. The difference between the original and re-calculated depth measurements after correction is less than those in Fig. 15a. Fig. 18 summarizes the sequence described above.

**Fig. 15.** Cumulative graph showing difference between area of distribution of LiDAR and GPR data before (a) and after (b) linear trend correction (Bridge Deck 1). (1 in. = 25.4 mm).

**Fig. 16.** Cumulative graph showing area of distribution versus concrete removal depth for every 0.5 in. (Bridge Deck 1). Reflection amplitude data calculated using linear trend equation and those with different percentage of error (±40%, ±30%, ±20%, ±10%) are compared with original LiDAR data (dashed line). (1 in. = 25.4 mm).
Fig. 17. Linear trends obtained before (white) and after (red) correction (Bridge Deck 1). (1 in. = 25.4 mm).

Fig. 18. Sequence diagram showing steps to develop relationship between GPR reflection amplitude and possible depth of concrete removal for Bridge Deck 1.
Using the procedure described above, the authors performed statistical analysis for Bridge Deck 2 (Figs. 19, 20, 21, and 22). The correlation coefficient in Fig. 19a \((r = -0.46)\) is slightly lower than that in Fig. 13a for Bridge Deck 1, however, the authors assumed a moderate linear relationship for this particular data set. It should be noted that a 90% error bar (Fig. 19b, Fig. 21) was used for category 0.5-4.6 to find a better match. This off-value might be an indication that a linear trend equation overestimates deterioration shown by GPR in the areas with no or little concrete depth removal.
Fig. 21. Cumulative graph showing area of distribution versus concrete depth removal for every 0.5 in. (Bridge Deck 2). Amplitude data either calculated using linear trend equation and those with different percentage of error (-50%, ±40%, ±30%, ±20%, ±10%, +90%) are compared with original LiDAR data (dashed line). (1 in. = 25.4 mm).

Fig. 22. Linear trends obtained before (white) and after (red) correction (Bridge Deck 2). (1 in. = 25.4 mm).
7. Discussion of results

Based on the analyses performed on the data from Bridge Deck 1 and Bridge Deck 2, it was determined that a linear relation between GPR data and depth of concrete removal can be established. The main advantage of using a linear relation between the GPR reflection amplitude data and the concrete removal thickness data is its ability to manipulate amplitude data within any range. Various amplitude ranges are expected from bridges with different deck design and/or investigations influenced by moisture presence, as water tends to develop significant signal attenuation and, therefore, weaken reflection amplitudes. Fig. 23 shows the two best-fit linear equations calculated for Bridge Deck 1 and Bridge Deck 2. It is noted that the slopes of the two equations are almost similar, but the y-intercepts are different. The difference in the slopes and intercepts is likely to be caused by several factors, such as different depth to the top mat of reinforcing steel, different weather conditions at which data were acquired, various concrete properties, etc.

![Linear Equations Graph](image)

**Fig. 23.** Best-fit linear trends showing relation between concrete depth removal and amplitude for Bridge Deck 1 and Bridge Deck 2. (1 in. = 25.4 mm).
Using the revised equations, the concrete repair quantities for three categories were re-computed and summarized in Table 3. As seen, LiDAR measurements and the repair re-computed quantities are very close (within 3%) after the linear equations were corrected.

<table>
<thead>
<tr>
<th></th>
<th>Bridge Deck 1</th>
<th></th>
<th>Bridge Deck 2</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LiDAR</td>
<td>Revised</td>
<td>LiDAR</td>
<td>Revised</td>
</tr>
<tr>
<td></td>
<td>measurements</td>
<td>equation</td>
<td>measurements</td>
<td>equation</td>
</tr>
<tr>
<td></td>
<td>(%)</td>
<td>(%)</td>
<td>(%)</td>
<td>(%)</td>
</tr>
<tr>
<td>Depth of concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>removed, in.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.75 and less</td>
<td>20.6</td>
<td>22.7</td>
<td>37.9</td>
<td>38.1</td>
</tr>
<tr>
<td>(19 and less)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.75 – 1.825</td>
<td>59.7</td>
<td>56.1</td>
<td>54.6</td>
<td>53.1</td>
</tr>
<tr>
<td>(19 – 46)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.825 and greater</td>
<td>19.7</td>
<td>21.2</td>
<td>7.5</td>
<td>8.8</td>
</tr>
<tr>
<td>(46 and greater)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

As shown in Section 6, data editing (or correction) is essential for the following reasons. Firstly, it removes the effect of overestimating the percentage of concrete removal depth shown by GPR when no evidence of deterioration is observed. The presence of this effect can be explained by the fact that the water pressure of the hydrodemolition equipment is set to remove a minimum thickness of concrete from sound zones. Secondly, data correction prevents underestimating the percentage of anomalies of extensive deterioration. This suggests that the hydrodemolition technique removes more concrete than shown by the initial linear trend equation. As reinforcing steel bar corrodes, it expands and cracks concrete all around the bar. Although GPR reflections are measured from the top of the reinforcing bars, extensive deterioration results within the concrete that is deeper than the upper layer of reinforcing bars. These
two reasons suggest that the slope of the trend line should be increased to provide a better estimate of concrete deterioration.

It should also be noted that perfect correlation between the GPR reflection amplitude and the concrete removal maps is not expected. The GPR responds to the presence of saline moisture present in the deck, whereas hydrodemolition removes weaker concrete. GPR and rehabilitation results are therefore expected to correlate best in those areas where the pore space within physically degraded concrete is infilled with slightly saline moisture. Apparent discrepancies between the GPR and concrete removal results could also be caused by the fact that the GPR maps reflect degradation and saline moisture within a thickness of concrete that was removed by milling prior to hydrodemolition. Furthermore, the GPR data are based on the reflection amplitudes from the top transverse layer of reinforcement and do not represent the condition of the concrete below the top transverse reinforcement. Therefore, the depth of concrete material removed beneath the top of reinforcing bars is not reflected directly in the GPR results. Thus, the two data sets do not have a direct physical correlation. Finally, the GPR maps were generated with 1 ft. (305 mm) and 2 ft. (610 mm) traverse spacings in this study, while the LiDAR survey mappings were produced with a much denser grid. Even though the authors used one type of interpolation for both mapping, the mapping accuracy is expected to be different.

8. Concluding remarks

The main objectives of this paper were to study and describe a possible relationship between GPR reflection amplitude and concrete removal depth after hydrodemolition. The pre-rehabilitation GPR condition assessments were validated by comparing the distribution of the top reinforcing bar reflection amplitudes with the post-hydrodemolition concrete removal depth, which showed a reasonable spatial correlation. Data from the GPR reflection amplitude maps and the survey maps of concrete removal depth were digitized to produce a scatter plot. The authors assumed, justified, and corrected a linear regression equation for each of the two case study decks that describes the relationship between reflection amplitude and concrete removal depth. The main
challenge was to determine the slope, which has a significant influence on repair quantity estimates, while the intercept would differ depending upon various factors such as different depth to the top mat of reinforcing steel, different weather conditions at which data were acquired, various concrete properties, etc. Results of the two case studies presented in this paper show a reasonable correlation between GPR reflection amplitude data results and depth of concrete removal during hydrodemolition. This work illustrates that GPR data has the potential to be used to predict concrete repair estimates. Further study is needed to advance this technique and potentially establish a single equation so that more accurate estimates of thickness of deteriorated concrete can be calculated based on GPR reflection amplitudes.

Acknowledgments

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References


III. Integrated approach in assessing bridge deck condition

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ABSTRACT

This paper presents an integrated approach to assess concrete bridge deck condition using multiple assessment techniques. Four techniques – visual inspection, GPR, USW, and core control - were used to perform a bridge deck assessment. The bridge deck was then rehabilitated, and LiDAR measurements of concrete depth removal collected after hydrodemolition were used as ground truth. Qualitative and quantitative comparisons of data collected using non-destructive and destructive techniques were performed in this study. The results suggest more reliable results will be obtained if multiple bridge deck assessment methods are employed because the different tools respond to different types of deterioration.
1. Introduction

Degradation in reinforced concrete structures such as bridge decks is a significant problem that can lead to serviceability problems and even structural failure. In order to prevent failure and extend the service life of concrete bridge decks, proper assessment must be done periodically so that potential problems are detected and addressed in a timely manner.

Effective bridge deck assessment methods are widely discussed in the literature. The most common techniques include chain drag, visual inspection, ground penetrating radar (GPR), infrared thermography [1-5]. Nowadays, it is common that assessment of a given bridge deck is limited to only one or two techniques. However, concrete degradation is a complex process that involves physical, chemical, and electrochemical processes, and no single technology is capable of detecting all types of deterioration. In this respect, employing multiple methods that are each responsive, in part, to different types of defects may be beneficial.

The objective of this paper is to assess different bridge deck assessment methods by evaluating the data collected during a case study investigation. To achieve the objective, four methods of bridge deck evaluation are described and compared in this study. In addition to the assessment methods, concrete removal data were collected after hydrodemolition are analyzed as ground truth. The differences are examined in a qualitative analysis and measured in a quantitative analysis.

2. Background

Various methods are employed to assess the condition of a bridge deck. Although each method has limitations, each can provide useful information. This section describes bridge deck evaluation methods that were employed in this study.

Visual inspection is the predominant bridge deck evaluation technique used to detect various types of surface distress (asphalt and concrete patches, cracks, unfilled spalls, potholes, etc.) resulting from either environmental or human actions [4, 5]. During
visual inspection, each type of deterioration is documented; photographs are frequently taken for later reference (Fig. 1). One advantage of visual inspection is that this method requires a minimum level of training and can be performed rapidly. The main disadvantage of the method is its inability to detect deterioration below the deck surface. Typically, visual inspection is used to determine if further more detailed assessment is warranted [4].

![Fig. 1. Bridge deck conditions observed during visual inspection.](image)

Core control has been widely used to compare and verify assessment data collected using non-destructive tools [1, 6, 7]. A core sample can provide both qualitative and quantitative information about the condition of the concrete at the specific core location. Different types of laboratory tests can be performed on suitable cores to determine specific characteristics (e.g. chloride-ion concentration, volume of permeable pore space, density, compressive strength, elastic modulus, etc.). The principle limitation of this method is that it cannot be used to assess the entirety of the bridge deck, as typically only a few cores are extracted from each deck. The acquisition of cores is expensive and inconveniences the public as extended lane closures are often required. For example, it took about 30 minutes to extract each core at the test site described in Section 3 of this paper.

Ground penetrating radar (GPR) is a nondestructive tool commonly used to assess condition of concrete bridge decks [8, 6, 4, 9, 10, 11, 5, 12]. GPR bridge deck data are
not necessarily indicative of the physical condition of the concrete or the reinforcing steel. Rather GPR data are usually indicative of the relative concentrations of saline moisture within the concrete. However, GPR data are normally interpreted with the expectation that variations in saline moisture content are indicative of variations in the physical integrity of the bridge deck and the condition of the encased reinforcing steel.

During GPR data acquisition, 2-D images of the concrete deck are displayed in real time. Initial data interpretations can be made in the field (Fig. 2). Indeed, core locations are often selected on the basis of the field interpretations of GPR data. The GPR tool emits pulses of radio wave frequency electromagnetic (EM) radiation and measures the amplitudes and travel times of the pulsed EM signals that have been reflected from reinforcing bars and the base of the bridge deck. The main advantage of the GPR tool is the relatively high speed of data collection which minimizes traffic disruption. The main disadvantage is that the GPR tool provides only indirect information about the integrity of the concrete and presence of corroded reinforcing steel.

Ultrasonic surface wave (USW) is an acoustic method that is commonly used for bridge deck assessment [3, 13, 9, 5]. At each test location, the USW tool outputs a 1-D plot of elastic modulus (Young’s modulus) (Fig. 3). The principal advantage of this tool is that the output elastic modulus is indicative of the physical condition of the concrete.

The principal limitation of this technology is that it cannot be used to measure the elastic modulus of the near-surface concrete, as elastic moduli cannot normally be measured confidently for depths shallower than 2 in. because the phase velocities of very high frequency surface waves (Rayleigh waves) cannot be reliably measured using this tool.

Results of a bridge deck assessment are used to determine whether (areal extent and depth) the bridge deck is in need of rehabilitation, and the process used to rehabilitate a bridge deck is determined based upon the deck condition. Extensively corroded bridge decks are usually the subject of complete deck replacement. Less deteriorated bridge decks may undergo removal of deteriorated concrete by using jackhammer, hydrodemolition, and/or milling. Following the removal, replacement of deteriorated concrete is performed by partial- or full-depth patching.
Fig. 2. A typical GPR scan collected on reinforced concrete bridge deck.

Fig. 3. A typical 1-D plot of elastic modulus.
3. Case study

3.1. Bridge deck description

The case study bridge was built in 1972. The main function of the bridge is to carry U.S. 50 east- and west-bound traffic over the Union Pacific Railroad. The bridge deck is a solid cast-in-place concrete slab supported on steel girders. According to the design drawings, the deck is 46 ft. - 10 in. wide, and the total structure length is 157 ft. The concrete deck thickness is 7.5 in., with the top layer of reinforcing steel oriented in the transverse direction of the bridge (perpendicular to traffic flow). The top reinforcing steel bars are located at a depth of 1.875 in. (to top of bars) and are spaced 5 in. center-to-center based on the design drawings.

3.2. Bridge deck assessment surveys

The surveys described in this section were carried out simultaneously on October 24, 2012. Approximately 3 in. of rain was observed in the area within 3 days prior to the fieldwork.

The investigations of the bridge deck were performed one lane at a time, with the other lane remaining open to traffic during data acquisition. The crew included 7 people. It took approximately 7 hours to conduct the initial non-destructive and destructive surveys.

3.2.1. Visual inspection

For this study, a thorough visual inspection of the top surface of the bridge deck was performed. Cracks, patches, and other anomalies were measured and documented. Notes taken from the visual inspection survey were incorporated into drawings of the bridge showing size, location, and type of the defect observed. During the visual investigation, 69 defects were documented as shown in Fig. 4. The main types of visible defects noted were cracks, concrete patches, and asphalt filled potholes. The majority of the patches were in fair condition. The deck also exhibited many cracks in the transverse direction (perpendicular to the direction of traffic flow).
In addition to the observations made from the deck surface, the underside of the deck was also examined. The underside examination revealed four areas with evidence of deterioration (Fig. 5). The approximate sizes and locations were plotted on the map of the bridge deck (Fig. 4).
To aid in the comparative assessment of the acquired bridge deck assessment data, each section of the deck was classified (qualitatively) as being in categories: good, fair or poor condition based on the visual inspection results. “Good” indicates no defects on either the top surface or underside of the deck were observed. A rating “Fair” was assigned to the areas where minor visual defects were present. “Poor” indicates areas with higher densities of defects. The classified areas of the bridge deck are mapped in Fig. 4.

3.2.2. Core control

In this study, 2 in. diameter core samples (approximately 4 in. in length) were acquired at locations selected on the basis of the field assessment of visual inspection and GPR data. Cores were acquired in areas where there was no evidence of deterioration and in areas where the bridge deck appeared to be deteriorated.

The bridge deck cores, in their entirety, were carefully examined and described (Fig. 6). Visible properties documented included diameter, surface material, number of pieces and the length of each piece, presence of reinforcing bar, concrete roughness, number of voids, quality of aggregate coating with the paste mixture in the concrete, the volume of paste, signs of air entrainment, flaking surfaces, discolorations, delaminations, segregation of the aggregate, and presence of cracks. Based on this qualitative analysis, the cores were assigned a rating of either “Good”, “Fair”, or “Poor” for the purpose of this study. A visual core rating of “Good” indicates neither delaminations nor visible deterioration were present. “Fair” indicates the core exhibited visible deterioration possibly including minor delaminations. “Poor” indicates that the core was extensively deteriorated and was recovered in multiple fragments [9]. Due to the size and condition of the cores, laboratory tests of compressive strength and elastic modulus were not conducted. Volume of permeable pore space was determined in accordance with ASTM C642 [14]. Core descriptions are summarized in Table 1.

In addition to the visual core inspection, USW data were collected at each core location before the core sample was extracted. The measured elastic moduli for each core sample are presented in Table 1.
**Fig. 6.** Core samples retrieved from the bridge deck.

**Table 1**  
Evaluation of cores.

<table>
<thead>
<tr>
<th>Core</th>
<th>A1</th>
<th>A2</th>
<th>A3</th>
<th>B1</th>
<th>B2</th>
<th>B3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (in.)</td>
<td>2.5 – 3.0</td>
<td>2.8 – 3.4</td>
<td>3.1 – 3.8</td>
<td>~3.0</td>
<td>5.3 – 6.0</td>
<td>3.9 – 4.1</td>
</tr>
<tr>
<td>Number of pieces</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td>5</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Roughness* (Smooth, Average, Very Rough)</td>
<td>Smooth</td>
<td>Average</td>
<td>Average</td>
<td>Rough</td>
<td>Average</td>
<td>Average</td>
</tr>
<tr>
<td>Delaminations: depth (in.)</td>
<td>None</td>
<td>None</td>
<td>1.875</td>
<td>1.5, 1.75, 2.0</td>
<td>1.0</td>
<td>None</td>
</tr>
<tr>
<td>General quality of concrete (good, fair, poor)</td>
<td>Good</td>
<td>Good</td>
<td>Fair</td>
<td>Poor</td>
<td>Fair</td>
<td>Good</td>
</tr>
<tr>
<td>Volume of permeable pore space (ASTM C642) (percent)</td>
<td>13.7</td>
<td>14.4</td>
<td>14.0</td>
<td>15.7</td>
<td>13.2</td>
<td>14.3</td>
</tr>
<tr>
<td>Elastic modulus (USW) (ksi)</td>
<td>2457</td>
<td>3986</td>
<td>3785</td>
<td>3238</td>
<td>1638</td>
<td>3102</td>
</tr>
</tbody>
</table>

*Evidence of loose or missing aggregate around the outside of the core.*
3.2.3. **Ground penetrating radar (GPR)**

In this study, GPR data were acquired across the bridge deck (along parallel traverses oriented perpendicular to direction of traffic) using a GSSI 1.5 GHz ground-coupled antenna mounted to a hand-pushed cart. Forty-two GPR profiles (spaced at 1 ft. intervals) were collected on the deck (6552 linear feet).

The GPR data were processed and analyzed using RADAN 6.5 [15]. During processing, the amplitudes of the reflection from the uppermost layer of reinforcing steel were normalized using the method proposed by Barnes et al. [16] and then plotted (Fig. 7).

![Fig. 7. GPR reflection amplitude rebar mapping with core and USW section locations superposed.](image)

To aid in the comparative analyses of the GPR data, amplitude ranges for different deterioration categories were defined based on the previous authors’ experience and visual examination of GPR scans. The visual evaluation involved identifying amplitude ranges for regions with and without evidence of deterioration. As a result, three categories were defined and include “Good”, “Fair”, and “Poor”. The GPR-based classification of the bridge deck is shown in Fig. 7.

3.2.4. **Ultrasonic surface wave (USW)**

USW testing was carried out using a Portable Seismic Property Analyzer (PSPA). PSPA consists of a source, two receivers, and an electronics box packaged as a hand portable unit. The PSPA operates with a laptop computer that is connected to the hand-
carried transducer unit by a cable. The PSPA utilizes both impact echo (IE) and USW techniques. In this study, however, the authors analyzed USW data only.

![Fig. 8. Elastic modulus contour mapping of Section A and Section B (Figure 7).](image)

During USW testing, the spacing between receivers was set at 4 in., which allowed for an investigation depth range of 2 to 7 in. The USW data were acquired from the top surface of the deck at discrete points spaced at 2 ft. intervals. Due to time constraints, USW data were collected at two areas of the bridge deck (Fig. 7), with 24 USW data sets being acquired in each area (Fig. 8). The automatically generated output at each test location was a 1-D plot of elastic modulus extending from a depth of approximately 2 in. to a depth of approximately 7 in. Maps are shown in Fig. 8. To aid in the comparative analyses of the USW data, a rating scale was developed based on published literature regarding the elastic modulus of concrete [17]. The initial rating included four categories: “Good”, “Fair”, “Poor”, or “Severe.” For the interpretation purposes of this paper, the authors combined “Poor” and “Severe” categories into one. Thus, a rating of “Good” indicates that the average elastic modulus was greater than or equal to 5000 ksi. “Fair” indicates that the average elastic modulus was in the range of 4000 - 5000 ksi. “Poor” indicates that the average elastic modulus was less than 4000 ksi.
3.3. Bridge deck rehabilitation

After the bridge deck inspection, the deck underwent rehabilitation process that included milling and hydrodemolition. During hydrodemolition, water jets with a constant pressure are used to remove deteriorated concrete from the top surface of the bridge deck [18], leaving the sound concrete in place. Typically, the hydrodemolition process also removes corrosion from exposed reinforcing steel and roughens the deck surface to provide adequate adhesion to the new overlay (Fig. 9).

![Bridge deck surface with exposed corroded rebar after removal of deteriorated concrete by hydrodemolition.](image)

LiDAR (Light Detection and Ranging) was used to map the surface of the bridge deck with the objective of estimating the thickness of material removed. LiDAR is a remote sensing technology that uses laser to measure distances [19]. LiDAR provides high-resolution mapping of surfaces, and is widely used for many research and engineering applications.

In this study, LiDAR measurements were made twice. The first measurements were taken from the original surface of the deck. The second measurements were taken after milling and hydrodemolition. A final map of LiDAR data (Fig. 10) was derived from the subtracting pre-rehabilitation and post-rehabilitation measurements, and therefore represents the thickness of concrete removed from the top surface of the deck.
In this respect, LiDAR mapping of concrete depth removal was used as ground truth data.

Based on the depth of material removal during hydrodemolition and condition of rebars exposed after hydrodemolition, three ratings were derived from the LiDAR mapping.

A rating of “Good” was assigned to a depth of removal less than 1.4 in., which is the depth of material removed by milling and hydrodemolition of sound concrete. A rating of “Fair” was assigned to material removal depths between 1.4 in. and 2.2 in. The depth of 2.2 in. was chosen on the basis of visual observations of areas where reinforcing bars appeared to be in fair condition (slightly corroded). A rating of “Poor” was assigned to material removal depths greater than 2.2 in. As for the reinforcing bar condition, extensively corroded bars were present in poor quality areas. The categories are shown in Fig. 10.

4. Comparative analyses of the acquired assessment data

Two types of comparative analyses are described in this section: qualitative and quantitative. The qualitative comparisons are based mostly on visual assessments of the
different data sets. The quantitative comparisons are based on the assessment of the quantifiable properties of the different data sets.

4.1. Qualitative comparisons

First, LiDAR measurements of concrete removal depth are compared with all the non-destructive data (visual survey, GPR, and USW). The comparisons help to evaluate the performance of each technique. Following, the non-destructive data sets are compared with each other. The commonalities and differences among the techniques help to understand the physical principles of each technique.

Comparison between LiDAR and GPR data

Visual analysis of the plotted GPR and LiDAR data (Fig. 7 and Fig. 10) indicates there is a good correlation between the two data sets. Areas designated as good on the GPR plots tend to be located in areas designated as good on the LiDAR mapping of concrete removal depth. Similarly, areas designated as poor on the GPR plots tend to be located in areas designated as poor on the LiDAR mapping.

This correlation is expected, as hydrodemolition removes mechanically weak concrete. Typically weak concrete is porous and permeable, and contains higher concentrations of saline moisture. In a recent study by Varnavina et al. [11] a linear relationship between GPR reflection amplitude and LiDAR measurements collected after hydrodemolition was established. As a result, a linear regression equation can be used to predict concrete repair estimates.

Comparison between LiDAR and visual inspection data

The LiDAR results show a reasonable correlation with the documented visual defects (Fig. 4 and Fig. 10). More specifically, good condition areas on the LiDAR mapping correspond well with good condition areas on visual inspection mapping. Conversely, areas of poor condition on the LiDAR mapping tend to be located where poor condition was determined by visual inspection.

It is worth noting, that hydrodemolition did not remove greater thickness of concrete in areas where transverse cracks were present. It should also be noted that no patches and/or potholes remained in place after hydrodemolition. The zones of underside deterioration revealed a greater thickness of concrete removed during hydrodemolition.
On the other hand, some areas with no underside deterioration showed significant depth of concrete removal (3 in. and more).

**Comparison between LiDAR and USW data**

A reasonable correlation between USW and LiDAR mappings is noticeable in Fig. 8 and Fig. 11. Good quality areas on LiDAR mapping correspond to fair quality areas on USW mappings. Fair quality areas on LiDAR mappings correspond to the areas with poor quality of concrete shown on USW mappings.

![LiDAR mappings of concrete removal depth generated for Section A and Section B (Fig. 7).](image)

Fig. 11. LiDAR mappings of concrete removal depth generated for Section A and Section B (Fig. 7).

Although LiDAR results are indicative of the quality of the upper 4 in. of concrete and USW results are indicative of the concrete between depths of 2 and 7 in., a similarity between the two data sets is not surprising, as elastic moduli indicate stiffness of concrete, while hydrodemolition removes mechanically weak concrete and leaves
mechanically strong concrete in place. A closer similarity would be expected if USW testing was performed on a denser grid, as opposed to 2x2 ft. grid cell.

Comparison between LiDAR and core data

To study the comparison between the LiDAR and core samples, 2x2 ft. sections (Fig. 12) were generated using the original LiDAR mapping so that the depth of material removed by hydrodemolition can be observed for the areas where cores were extracted. The comparison between the two is summarized in Table 2.

![Core locations on LiDAR mapping](image)

**Fig. 12.** Core locations on LiDAR mapping (dimension of each section is 2x2 ft; contours are in units of in.).

According to the visual evaluation, core A1 was rated good and revealed no signs of deterioration. Approximately 1.8 in. of concrete was removed by the hydrodemolition in the area where core A1 was extracted. Similar to A1, core A2 did not reveal deterioration during the visual evaluation. However, more concrete (2.6 in.) was removed from the area where core A2 was extracted. Core A3 appeared to be in fair condition, having a single delamination at the depth of 1.875 in. (Fig. 6). LiDAR measurements showed approximately 1.3 in. of concrete removal in the area where core A3 was retrieved. Core B1 was rated poor during visual evaluation. Approximately 1.7 in. of material was removed at the location in close proximity to core B1. Similarly to core B1, LiDAR measurements collected in proximity to core B2 showed 1.7 in. of material removed. Based on the visual evaluation, core B2 was rated fair. Core B3 was rated good during the visual evaluation. Roughly 1.4 in. of material was removed from the area where core B3 was extracted.
Table 2
USW, GPR, LiDAR, and visual inspection results at core locations are compared with results visual core evaluation.

<table>
<thead>
<tr>
<th></th>
<th>A1</th>
<th>A2</th>
<th>A3</th>
<th>B1</th>
<th>B2</th>
<th>B3</th>
</tr>
</thead>
<tbody>
<tr>
<td>USW (good, fair, poor)</td>
<td>Poor</td>
<td>Poor</td>
<td>Poor</td>
<td>Poor</td>
<td>Poor</td>
<td>Poor</td>
</tr>
<tr>
<td>GPR (good, fair, poor)</td>
<td>Good</td>
<td>Fair</td>
<td>Fair</td>
<td>Fair</td>
<td>Fair</td>
<td>Fair</td>
</tr>
<tr>
<td>Visual inspection (good, fair, poor)</td>
<td>Fair</td>
<td>Fair</td>
<td>Fair</td>
<td>Poor</td>
<td>Fair</td>
<td>Poor</td>
</tr>
<tr>
<td>Core inspection (good, fair, poor)</td>
<td>Good</td>
<td>Good</td>
<td>Fair</td>
<td>Poor</td>
<td>Fair</td>
<td>Good</td>
</tr>
<tr>
<td>Hydrodemolition (good, fair, poor)</td>
<td>Fair</td>
<td>Poor</td>
<td>Good</td>
<td>Fair</td>
<td>Fair</td>
<td>Good</td>
</tr>
</tbody>
</table>

In summary, poor correlation was identified between visual core evaluation results and LiDAR measurements of concrete removal. Such observations may partly arise from the fact that visual core evaluation does not provide information on concrete strength, while hydrodemolition is directly related to the strength of material.

**Comparison between GPR and core data**

The cores are compared in terms of the visual core rating to the GPR deterioration level estimated for the core location. The comparison between the two is summarized in Table 2. A good correlation between the two data sets was found for two cores A3 and B2. More specifically, cores A3 and B2 were rated fair and were taken from the areas where GPR points fair and poor condition, respectively. On the other hand, the correlation between the two is not good for cores A1, A2, B1, and B3.

It should be noted, however, that perfect correlation between the two was not expected, because GPR responds to the presence of saline moisture, while visual core inspection evaluate general core condition. In order to support the correlation, measurements of chloride ion concentration should be taken in addition to the visual core evaluation. Additionally, interpolation between the GPR traverses spaced at 1 ft. may have a misleading effect on the GPR data near the core location, especially when the results are being compared to a relatively small 2 in. diameter core.
Comparison between GPR and visual inspection data

As seen in Fig. 4 and Fig. 7, a good correlation between the two data sets is identified. Good quality on the GPR mapping tend to be located where visual inspection revealed no signs of deterioration on the top surface of the deck. In contrast, poor quality areas identified by GPR correspond to poor quality areas on the visual inspection mapping.

This relationship is expected for either or both of the following reasons. Firstly, new concrete or/and asphalt overlay has different electrical properties than the existing material. The electrical properties of material (dielectric permittivity and electrical conductivity) control the velocity and attenuation of EM signal. As a result, amplitude anomalies may be associated with patch material, as opposed to probable corrosion. Secondly, the patched area may not have been repaired properly so that moisture penetrated through the interface and initiated rebar corrosion.

Comparison between core control and USW data

In addition to the visual core examinations, USW data were acquired in immediate proximity to the core locations. The cores are compared in terms of the visual core rating to the USW deterioration level estimated for the core location. The comparison between the two is summarized in Table 2.

Three cores (A3, B1, B2) showed a reasonable match, while the other three (A1, A2, B3) showed a contrasting match. The source of possible discrepancies is that the average elastic moduli were calculated for a depth range of ~2-7 in., while cores did not exceed 6 in. in length. Additionally, no measurements of core strength or modulus were taken after they were extracted to support the comparison.

Comparison between visual inspection and USW data

As shown in Fig. 8 and Fig. 13, areas designated as fair on the visual inspection mappings are located where the USW measurements determined fair and poor quality. Good quality areas, however, were not identified on the USW mappings, but were present on the visual inspection mapping.

In summary, the USW data resulted in a fair agreement with visual inspection. This agreement, however, leads to misinterpreting anomalies associated with differences
in mechanical properties of material, but not deterioration. Thus, a careful visual evaluation of repair condition should be performed before USW data interpretation.

Comparison between visual inspection and core data

The cores are compared in terms of the visual core rating to the visual inspection deterioration level estimated for the core location. The comparison between the two is summarized in Table 2.

For this comparison, the ideal match would be a core rated good during the visual evaluation to be extracted from an area of good quality based on visual inspection; a core rated fair to be extracted from an area with fair quality; and a core rated poor to be extracted from an area with poor quality. Thus, the ideal match was found for three cores (A3, B1, B3). The other three core samples showed some differences.

As visual inspection evaluates surface conditions, while core control is intended to determine condition inside the deck, the correlation between the two is fair.
**Comparison between GPR and USW data**

To determine correlation between the GPR and USW results, GPR reflection amplitude mappings were generated for two sections where USW data were acquired. The resulting maps are shown in Fig. 14.

![Reflection amplitude mappings for Section A and Section B](image)

**Fig. 14.** Reflection amplitude mappings generated for Section A and Section B (Fig. 7).

Although USW targets a depth of 2-7 in., and GPR is intended to determine condition of the upper zone of the deck (up to 2 in.), the maps (Fig. 8 and Fig. 14) are somewhat similar. More specifically, good quality areas identified by GPR are located where USW identified fair quality areas. Fair quality areas shown on the GPR are located in poor quality areas identified by USW. This agreement between the two suggests that the condition of concrete in depth range of 2-7 in. to a certain extent is indicative of a condition in an upper layer.

**4.2. Quantitative comparisons**

To perform quantitative data comparison, each deterioration category was encoded numerically. Thus, number 1 was assigned to ‘good’, number 2 was assigned to
'fair', and number 3 was assigned to ‘poor’. Similar to the qualitative comparison, a separate analysis was performed for each pair of data sets. For the purpose of the comparison, Equation 1 was used to determine the correlation between the two data sets.

\[
\alpha = \frac{\sum_{i=1}^{n} |Method_1 - Method_2|}{n}
\]

where \(\alpha\) is the average correlation value, and \(n\) is the number of data points.

The closer the average value \(\alpha\) to 0, the better the correlation between the two. The comparisons are summarized in Table 3.

As seen in Table 3, the best numerical correlation was observed for the pair of GPR and LiDAR data collected after hydrodemolition. A good agreement between the two is expected, as hydrodemolition removes mechanically weak concrete that is porous and permeable, and contains higher concentrations of saline moisture. GPR, in turn, responds to the presence of saline moisture. The weakest correlation is observed for the pair of USW and visual assessment of cores. This could be explained by the fact that USW determines condition inside the bridge deck, whereas visual inspection is utilized for surface bridge deck conditions.

**Table 3**
Quantitative comparison between the assessment methods.

<table>
<thead>
<tr>
<th>Method 1</th>
<th>Method 2</th>
<th>Number of data points (n)</th>
<th>Average value ((\alpha))</th>
</tr>
</thead>
<tbody>
<tr>
<td>LiDAR</td>
<td>GPR</td>
<td>249</td>
<td>0.31</td>
</tr>
<tr>
<td>LiDAR</td>
<td>Visual inspection</td>
<td>249</td>
<td>0.55</td>
</tr>
<tr>
<td>LiDAR</td>
<td>USW</td>
<td>136</td>
<td>0.95</td>
</tr>
<tr>
<td>LiDAR</td>
<td>Core control</td>
<td>6</td>
<td>0.67</td>
</tr>
<tr>
<td>GPR</td>
<td>Core control</td>
<td>6</td>
<td>0.50</td>
</tr>
<tr>
<td>GPR</td>
<td>Visual inspection</td>
<td>249</td>
<td>0.53</td>
</tr>
<tr>
<td>Core control</td>
<td>USW</td>
<td>136</td>
<td>1.33</td>
</tr>
<tr>
<td>Visual inspection</td>
<td>USW</td>
<td>136</td>
<td>0.95</td>
</tr>
<tr>
<td>Visual inspection</td>
<td>Core control</td>
<td>6</td>
<td>0.67</td>
</tr>
<tr>
<td>GPR</td>
<td>USW</td>
<td>6</td>
<td>0.95</td>
</tr>
</tbody>
</table>
5. Conclusions

This paper assessed different bridge deck assessment methods by evaluating the data collected during a case study investigation. Four techniques, namely visual inspection, GPR, USW, and core control, were used to perform a bridge deck assessment. The bridge deck was then rehabilitated, and LiDAR measurements of concrete depth removal collected after hydrodemolition were used as ground truth. Qualitative and quantitative comparisons of data collected using non-destructive and destructive techniques were performed in this study. The qualitative comparative analysis of data provided in-depth understanding of each method employed for the bridge deck assessment. More specifically:

- The LiDAR data showed a reasonable correlation with GPR and visual inspection data for the majority of the deck; the correlation with the USW data is also noticeable; the correlation between LiDAR and cores is less noticeable.
- As GPR and USW data are influenced by visual defects, visual inspection is essential to determine surface condition of the bridge deck. The information about bridge deck surface condition can be used to differentiate anomalies associated with deterioration from those associated with repairs.
- The coring data are used to complement, validate and support non-destructive survey data, but a large number of core samples are required to verify findings from non-destructive evaluation.
- The USW data provide information about concrete degradation, while GPR is sensitive to the corrosion of upper reinforcing steel layer. The investigation depth of USW testing is the range of 2-7 in., while GPR is intended to determine condition of the upper layer of the deck (e.g. top layer of reinforcing bars). Using both techniques, a full depth thickness deck assessment can be conducted.

The quantitative analysis of data provided numerical evidence of the agreement among the methods. The best correlation was observed between data collected after hydrodemolition and GPR. The weakest correlation was observed between USW and visual inspection of cores.
To summarize, the qualitative analysis of data suggests that integrated use of GPR, USW, coring, and visual inspections allows highlighting anomalies, which correspond to different types and stages of deterioration. The quantitative analysis is used to quantify the differences observed in the qualitative analysis.

Acknowledgments

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References


SECTION

2. CONCLUSIONS

The first paper described data acquisition and processing parameters that can be used for concrete bridge decks condition assessment using ground-coupled GPR system. Ground-coupled GPR system is typically used for detailed bridge deck investigations. The main consideration in using a ground-coupled GPR antenna is that it requires a significant amount of time for data acquisition and therefore causes traffic disruption. In order to reduce the time and cost of bridge deck inspections, appropriate data acquisition and processing parameters were offered.

The second paper presented a possible relationship between GPR reflection amplitude and concrete removal depth collected after hydrodemolition. A linear relationship between the two was assumed, justified, and corrected for each of the two case study decks. This relationship can be used as a rough guide to estimate concrete repair quantities on the basis of GPR reflection amplitude data.

The third paper presented an integrated approach in assessing bridge deck condition. The set of non-destructive and destructive was used to detect and characterize various types and levels of deterioration. Qualitative and quantitative comparisons of data collected by non-destructive and destructive techniques were performed in this study and suggested using multiple bridge deck assessment methods to reveal different signs of possible deterioration.
VITA

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