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Using Advanced Technologies to Assess Deterioration in Concrete Bridge Decks

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USING ADVANCED TECHNOLOGIES TO ASSESS DETERIORATION IN CONCRETE BRIDGE DECKS

BY

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ABSTRACT

Bridges throughout the nation are continuously deteriorating, and current improvement efforts have not been sufficient for closing the investment gap. Without investing in bridge condition today, the structural integrity of the bridges, as well as the comfort, cost, and most importantly safety of motorists is compromised. During routine bridge deck inspection, simplistic methods for assessing deterioration in concrete bridge decks are substandard and only capable of detecting deterioration in its moderate to severe stages. To provide a more thorough assessment of deterioration in concrete bridge decks, advanced technologies should be incorporated into bridge inspection. Using advanced technologies like surface roughness and ground penetrating radar, deterioration hidden from the naked eye or missed using traditional assessment methods can be more accurately detected, evaluated, and reported. When accurately reported, present condition can be compared to past condition to determine what improvement efforts should be made and when. Maintaining bridges in good condition presently is more cost-effective than rehabilitating or replacing bridges in poor condition in the future.

This study aims to demonstrate that a more thorough assessment of surface and subsurface deterioration in Rhode Island concrete bridge decks can be obtained through the use of advanced technologies like surface roughness and ground penetrating radar. Three Rhode Island concrete bridge decks, visually in good, moderate, and poor condition, are initially tested to generate surface and subsurface deterioration maps, then tested a second time 2 years later (the length of time of a typical routine bridge inspection) to study the change in

subsurface condition over time. Both initial and secondary findings are compared to reported bridge inspection deck conditions to assess accuracy in reported bridge deck condition. The subsurface conditions of the original test in 2015 will be compared to those of the secondary test in 2017, to determine change in subsurface condition over time using mean attenuation. Change in mean attenuation over time allows for the determination of rate of deterioration without the need for corroborative testing and without using a deterioration threshold. It is important to obtain a full picture of surface and subsurface deterioration, to determine rate of deterioration, and to accurately report findings during routine bridge inspection, to best determine what management strategies should be implemented and when, for preservation purposes.

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INTRODUCTION

1.1 The Nation's Infrastructure

Throughout the nation, vital infrastructure is continuously deteriorating, affecting the nation as a whole and each individual who uses this infrastructure on an everyday basis. For more than three decades, the World Economic Forum's annual Global Competitiveness Reports have studied and benchmarked the many factors underpinning national competitiveness, including infrastructure, technological readiness, and innovation. Technological advancement and breakthroughs have been at the basis of many of the productivity gains that our economies have historically experienced. Transforming not only the way things are being done, but also opening a wider range of new possibilities in terms of products and service, aid in maintaining a competitive edge[45].

In the 2017-2018 Global Competitiveness Report, the United States ranks 2^{nd} overall, below Switzerland, out of 137 countries for Global Competitiveness Index. Although ranking high overall, the United States ranks 25^{th} in basic requirements, and ranks 9^{th} in infrastructure [45]. Within the infrastructure component, the U.S. maintains a higher ranking because it finished 1^{st} in one subcategory: number of available airline seats. For quality of overall infrastructure, quality of roads, and quality of railroad infrastructure, the U.S. is ranked 10^{th}. For quality of port infrastructure, and quality of air transport infrastructure, the U.S. is ranked 9th [45]. Though the U.S. ranks 6th in technological readiness and 2nd in innovation, quality of infrastructure can be improved upon.

1.2 State of Overall Infrastructure

In addition to analyzing how the overall infrastructure of the United States compares to other competing countries, it is important to further distinguish how each infrastructure subcategory performs. In 1988, the concept of a report card to grade the nation's infrastructure was established by the American Society of Civil Engineers (ASCE). Specific categories included in infrastructure are aviation, bridges, dams, drinking water, energy, hazardous waste, inland waterways, levees, ports, public parks, rail, roads, schools, solid waste, transit, and wastewater. Eight criteria are used to determine grades for each category, including capacity, condition, funding, future need, operation and maintenance, public safety, resilience, and innovation [2].

Using the eight criteria, the American Society of Civil Engineers' Report Card for America's Infrastructure depicts the condition and performance of American infrastructure in the familiar form of a school report card – assigning letter grades based on the physical condition and needed investments for improvement [2]. When first originated in 1988, the nation's infrastructure earned a C, representing an average grade based on the performance and capacity of existing public works. Among the problems identified in this report were increasing congestion and deferred maintenance and age of the system; the authors of the report worried that fiscal investment was inadequate to meet the current operations costs and future demands on the system. In each of ASCE's

Report Cards, the Society found that these same problems persist [34]. In the 2017 ASCE Report Card for America's Infrastructure, infrastructure across the nation earned an overall D+ grade, entailing poor or at risk conditions [2].

CATEGORY	1988*	1998	2001	2005	2009	2013	2017
Aviation	B-	C-	D	D+	D	D	D
Bridges	–	C-	C	C	C	C+	C+
Dams	–	D	D	D+	D	D	D
Drinking Water	B-	D	D	D-	D-	D	D
Energy	–	–	D+	D	D+	D+	D+
Hazardous Waste	D	D-	D+	D	D	D	D+
Inland Waterways	B-	–	D+	D-	D-	D-	D
Levees	–	–	–	–	D-	D-	D
Ports	–	–	–	–	–	C	C+
Public Parks & Recreation	–	–	–	C-	C-	C-	D+
Rail	–	–	–	C-	C-	C+	B
Roads	C+	D-	D+	D	D-	D	D
Schools	D	F	D-	D	D	D	D+
Solid Waste	C-	C-	C+	C+	C+	B-	C+
Transit	C-	C-	C-	D+	D	D	D-
Wastewater	C	D+	D	D-	D-	D	D+
GPA	**C**	**D**	**D+**	**D**	**D**	**D+**	**D+**
Cost to Improve**	–	–	**$1.3T**	**$1.6T**	**$2.2T**	**$3.6T**	**$4.59T**

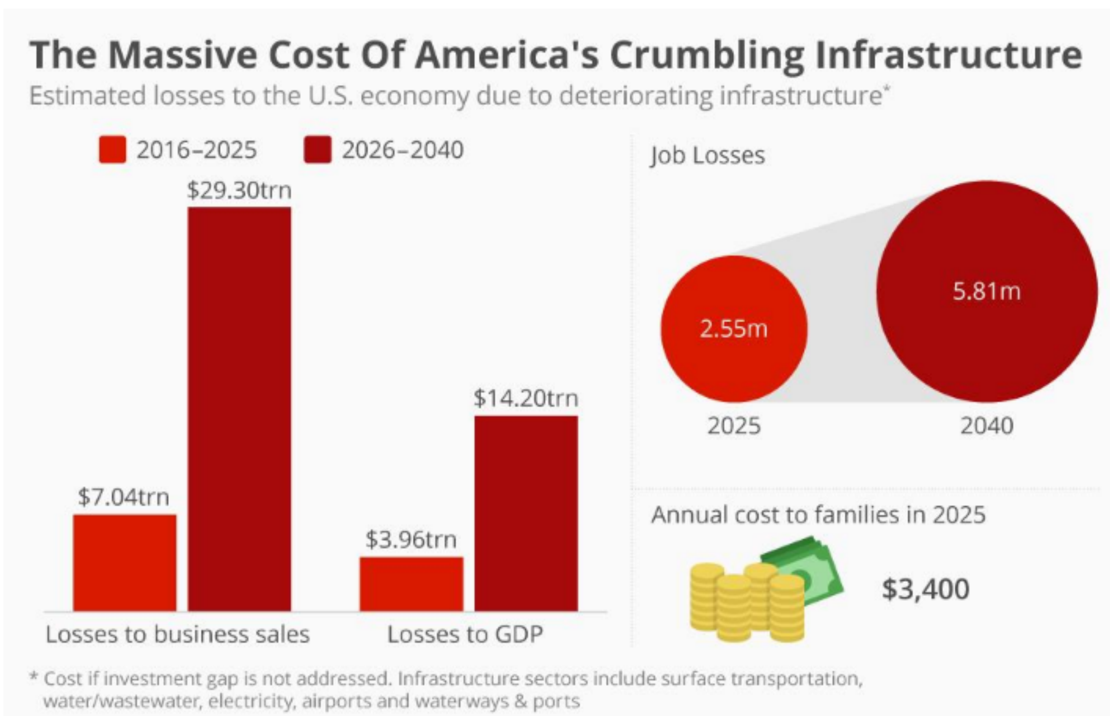
*The first infrastructure grades were given by the National Council on Public Works Improvements in its report Fragile Foundations: A Report on America's Public Works, released in February 1988. ASCE's first Report Card for America's Infrastructure was issued a decade later.

**The 2017 Report Card's investment needs are over 10 years. The 2013 Report is over eight years. In the 2001, 2005, and 2009 Report Cards the time period was five years.

Source: American Society of Civil Engineers

Figure 1.2.1: ASCE Report Card for America's Infrastructure History

As evident from past infrastructure reports, though efforts have been made to improve or better maintain condition over the years, infrastructure is continuously deteriorating, resulting in increased improvement costs with time. What was once a $1.3 trillion improvement cost in 2001, is now 3.5 times greater at $4.59 trillion in 2017, just 16 years later [34]. According to another report from the American Society of Civil Engineers, the U.S. economy is expected to lose just under $4 trillion in Gross Domestic Product (GDP) between 2016 and 2025 due to deteriorating infrastructure if investment gaps are not addressed [12].



Source: American Society of Civil Engineers

Figure 1.2.2: Cost of America's Infrastructure

This could hit $14 trillion by 2040 if the nation's aging roads, railways and bridges are left to decay even further. The report estimates that losses to business sales will amount to $7 trillion by 2025 while by 2040, they could soar as high as $23.3 trillion. Crumbling infrastructure will also have a knock-on effect on U.S. families' disposable household income. Between 2016 and 2025, each American household will lose $3,400 every year due to infrastructure deficiencies. The severe economic impact mentioned above will also cost some 2.5 million jobs by 2025, according to the report. Without investment, that number should reach 5.8 million by 2040 [12].

Most often, smaller and more continuous maintenance efforts in the present are much less expensive than larger and more complicated rehabilitation or replacement efforts in the future. It is therefore important not only to be able to detect deterioration in advance, but also to implement smarter, smaller, and more cost-effective management strategies as soon as needed in order to prevent significant deterioration in the future. Deterioration of infrastructure, especially bridges and roadways, affects the nation in its entirety, as each user of this infrastructure is affected in terms of comfort, cost, and most importantly safety. Our nation’s infrastructure is aging, underperforming, and in need of sustained care and action [34].

1.3 Condition Rating System for Bridges

In attempts to arrest the unremitting deterioration of America’s infrastructure, continuous maintenance and improvement efforts are needed to preserve present good condition rather than to replace future poor condition. For bridges, routine bridge inspections are performed in order to evaluate structural integrity and ensure that bridges remain safe for all users. When agencies inspect and maintain their bridges, unsafe conditions can be addressed and the possibility of closure minimized. A routine, or periodic, inspection is one of the many regularly scheduled inspections of a bridge that serves to evaluate the physical and functional conditions of the structure as compared to the initial or previously recorded conditions. Routine inspections help to ensure that all present service requirements are satisfied [14].

In most cases, routine inspections are required at least every two years. The bridge substructure, superstructure, and deck is evaluated, and any deficiencies are recorded. In addition, any necessary updates, additions, and/or corrections are made to the Structure Inventory and Appraisal Sheet [38]. The Structure Inventory and Appraisal Sheet verifies the safety of a bridge, in accordance with the National Bridge Inspection Standards (NBIS) and Department standards, and includes information regarding bridge identification, inspection, condition, load rating and posting, geometric data, age and service, structure type and materials, appraisal, classification, proposed improvements, and navigation data, along with bridge inspection deficiency notes.

A bridge may be classified as functionally obsolete or structurally deficient based on inspection findings. Functionally obsolete is a status used to describe a bridge that is no longer by design functionally adequate for its task. Reasons for this status include that the bridge doesn't have enough lanes to accommodate the traffic flow, it may be a drawbridge on a congested highway, or it may not have space for emergency shoulders. Functionally obsolete does not communicate anything of a structural nature. A functionally obsolete bridge may be perfectly safe and structurally sound, but may be the source of traffic jams or may not have a high enough clearance to allow an oversized vehicle [1]. Structurally deficient is a status used to describe a bridge that has one or more structural defects that require attention. This status does not indicate the severity of the defect but rather that a defect is present. The structural evaluation and the condition ratings of each bridge deck, substructure, and superstructure detail the nature and severity of the defect(s) [1].

During routine bridge inspections, both the quantity and the severity of each deficiency is noted in the inspection report, and an overall condition rating for each item (deck, substructure, superstructure) is given, as well as an overall structure condition rating. Overall structure condition ratings are used to describe the existing, in-place bridge as compared to the as-built condition. The Federal Highway Administration (FHWA) general structure condition ratings are given as follows [27]:

Table 1.3.1: FHWA Condition Rating Coding

Code	Description
N	Not applicable
9	Excellent condition
8	Very good condition - no problems noted
7	Good condition – some minor problems
6	Satisfactory condition – structural elements show some minor deterioration
5	Fair condition – all primary structural elements are sound but may have minor section loss, cracking, spalling, or scour
4	Poor condition – advanced section loss, deterioration, spalling or scour
3	Serious condition – loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	Critical condition – advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	Imminent failure condition – major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	Failed condition – out of service, beyond corrective action

Source: United States Department of Transportation/Federal Highway Administration

When provided with routine bridge inspection and updated Structure Inventory and Appraisal information, management agencies are able to continuously monitor changing bridge condition by comparing current bridge condition to previously recorded condition. Being able to effectively monitor bridge deterioration enables these agencies to better determine the most appropriate time to make easier improvement efforts before minor deterioration becomes much more significant. In addressing deterioration in advance, bridge preservation is a feasible and more practicable option rather than extensive repair or replacement.

Bridge deck inspection is a vital part of routine bridge inspections. Typically visually inspected, concrete bridge decks are examined for cracking, scaling, spalling, leaching, chloride contamination, potholing, delamination, and full or partial depth failures. These deficiencies include hairline, map, longitudinal, and transverse cracking, potholes, corrugation, and depressions, exposed, rusted and/or debonded rebar, rust staining and efflorescence, concrete discoloration, spalling, scaling, rutting, shoving, abrasion, and erosion. Deficiencies are noted in terms of quantity and severity, from which an overall deck condition rating can be determined. Deck condition rating specified by the National Bridge Inventory (NBI) for the Federal Highway Administration (FHWA) is as follows[27]:

Table 1.3.2: NBI Bridge Deck Condition Coding

Code	Description
N	Use for all culverts
9	Excellent condition – no noticeable or noteworthy deficiencies which affect the condition of the deck item. Usually new decks.
8	Very good condition – minor transverse cracks with no deterioration, i.e. delamination, spalling, scaling or water saturation
7	Good condition – sealable deck cracks, light scaling (less than ¼" depth). No spalling or delamination of deck surface but visible tire wear. Substantial deterioration of curbs, sidewalks, parapets, railing or deck joints (need repair). Drains or scuppers need cleaning.
6	Satisfactory condition – medium scaling (¼" to ½" in depth). Excessive number of open cracks in deck (5 ft intervals or less). Extensive deterioration of the curbs, sidewalks, parapets, railing or deck joints (requires replacing deteriorated elements).
5	Fair condition – heavy scaling (½" to 1" in depth). Excessive cracking and up to 5% of the deck area is spalled; 20-40% is water saturated and/or deteriorated. Disintegrating of deck edges or around scuppers. Considerable leaching through deck. Some partial depth failures, i.e. rebar exposed (repairs needed).
4	Poor condition – more than 50% of the deck area is water saturated and/or deteriorated. Leaching throughout deck. Substantial partial depth failures (replace deck soon).
3	Serious condition – more than 60% of the deck area is water saturated and/or deteriorated. Use this rating if severe or critical signs of structural distress are visible and the deck is integral with the superstructure. A full depth failure or extensive partial depth failures (repair or load post immediately).
2	Critical condition – some full depth failures in the deck (close the bridge until the deck is repaired or holes covered).
1	Imminent failure condition – substantial full depth failures in the deck (close the bridge until deck is repaired or replaced)
0	Failed condition – extensive full depth failures in the deck (close bridge until the deck is replaced).

Source: United States Department of Transportation/Federal Highway Administration

When a concrete bridge deck begins to deteriorate, it is important to make smaller and more cost-effective repairs before the deterioration reaches the deck reinforcement. Once deck reinforcement is exposed to water or salt due to surface cracking or spalling, corrosion ensues and a cycle of deterioration between the deck surface and subsurface begins. It is therefore crucial to better monitor deck condition in order to make the necessary improvement efforts before issues become bigger problems. When deck condition is better monitored, it must also be better reported, so that deterioration can be addressed and resolved before it is too late.

1.4 U.S Bridge and Road Condition

Over the past decade, there has been increased awareness of the significance of bridges to our nation's economy and the safety of the traveling public [4].Throughout the nation there are a total of 614,387 bridges that serve as vital links for means of transportation across the country. Though these hundreds of thousands of bridges are essential for transportation, they have received only a grade of C+ in the 2017 Report Card for America's Infrastructure, a grade entailing mediocre to adequate condition and capacity. Though most bridges are designed to last fifty years before major overhaul or replacement, the average age of an American bridge is well past middle age, at forty-three years, where almost four out of every ten, or 39%, of bridges are 50 years or older[4].

Amongst these bridges, approximately 55,910 bridges, or 9.1%, are rated as structurally deficient [4].In the nation's one hundred largest metropolitan areas alone, there are more structurally deficient bridges than there are McDonald's restaurants in the entire country. Laid end to end, all of the country's deficient bridges would span more than 1,500 miles, from Washington, DC to Denver, Colorado, or farther than from Canada to Mexico [8].Of these structurally deficient bridges, the average age is approximately sixty-five years old, or only twenty-two years older than the average bridge. Thus it is predicted for the future that in just ten years, one in every four bridges will be over the age of sixty-five, and will be far more likely to be deficient. This describes an additional 170,000 bridges becoming structurally deficient within the next ten years alone, due to the effects of age [5].

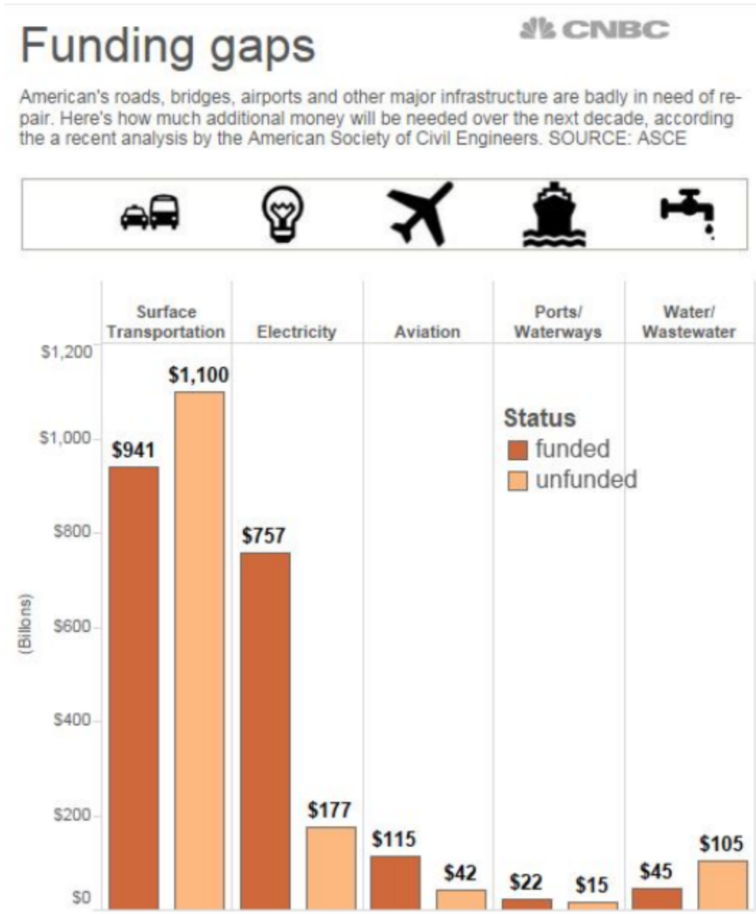
In a recent study, the Federal Highway Administration (FWHA) estimated that to eliminate the nation's bridge deficient backlog by the year 2028, we would need to invest $20.5 billion annually, while only $12.8 billion is being spent currently [7].The latest estimate put the nation's backlog of bridge rehabilitation needs at $123 billion. In recent years, investment at all levels of government has prioritized fixing bridges. Despite the increases in spending, investments in the country's bridges are insufficient [8].

America's roads are often crowded, frequently in poor condition, chronically underfunded, and are becoming more dangerous [39]. Though essential for transportation, roads have received a low grade of D in the 2017 Report Card for America's Infrastructure, a grade that entails poor condition and capacity, dropping from D+ in 2013. Similar to bridge decks, the condition of roadways is largely based upon deterioration and surface deficiency. When studying these effects on road condition and capacity, vehicular damage and cost, vehicular restrictions, and most importantly road user safety, are some issues of great concern.

Currently, 32% of America's major roads are in poor or mediocre condition, costing U.S. motorists $67 billion a year, or $324 per motorist, in additional vehicle repairs and operating costs [40]. Current estimates show that 42% of America's major urban highways are congested, resulting in 1.9 billion gallons of wasted gasoline and an average of 34 hours per year in traffic, costing the U.S. economy $101 billion [40]. Only one year later, more than two out of every five miles of America's urban interstates are congested and traffic delays cost Americans 6.9 billion hours delayed in traffic, or 42 hours per driver, thus wasting 3.1 billion gallons of fuel and costing a total loss of $160 billion [39].

Additionally, and perhaps most importantly, public safety is of great concern in regards to deficient pavement and roads in critical condition. Statistics indicate that roadway conditions are a significant factor in approximately one-third of all U.S. traffic fatalities, costing the U.S. economy $230 billion each year [40]. In a 2006 report, road conditions contributed to crash frequency or severity in 5.32 million crashes, or 31.4% of all traffic crashes nationally that year. Road condition related crashes accounted for 38.2% of non-fatal injuries (2.2 million cases), and 52.7% of fatalities (22,455 deaths). Bad design and conditions contributed to more deaths than speeding, drunken driving, or failure to use seatbelts [54]. After years of decline, traffic fatalities increased by 7% from 2014 to 2015[43].

Estimates state that to maintain the entirety of the nation's highways at their current condition would cost $101 billion, and in order to improve the nation's highways, investment would need to raise an additional $79 billion annually [42]. The ultimate cost of poor road conditions is significantly more over time than the cost to maintain those same roads in good condition. For example, after 25 years the cost per lane mile for reconstruction can be more than three times the cost of preservation treatments over the same period, which can lead to a longer overall lifespan for the infrastructure [40]. Current investment trends are doing little to improve roadway conditions and as a result, there is a decrease in condition and performance. With each passing year, the economic cost of underfunding maintenance and repair produces a mounting burden on our economy and increases costs to make improvements [40].



Source: American Society of Civil Engineers

Figure 1.4.1: Infrastructure Funding Gaps

The ASCE reports that within America's 2016-2025 infrastructure needs, surface transportation including bridges is the largest contributor, with only 46% funded ($941 billion), and $1.1 trillion underfunded. Our nation's infrastructure bill is overdue and costing every American family $9 each day [12]. The Federal Highway Administration estimates that each dollar spent on road, highway, and bridge improvements returns $5.20

in the form of lower vehicle maintenance costs, decreased delays, reduced fuel consumption, improved safety, lower road and bridge maintenance costs, and reduced emissions as a result of improved traffic flow [41].

1.5 Bridges and Roads in Rhode Island

In Rhode Island, both bridges and roadways are in critical condition. Though Rhode Island is the smallest state in the country, it is considered to be the worst ranked state in terms of bridge and road condition [36]. In 2013, of the 757 total bridges in Rhode Island, 156 or 20.6% of these bridges were considered to be structurally deficient, and 255 or 33.7% of these bridges were considered to be functionally obsolete [37]. In 2017, just four years later, the total number of bridge in Rhode Island increased to 772 bridges, with 24.9% deemed structurally deficient [36].Of the fifty states taken into consideration, the smallest state of Rhode Island ranks number twenty-one on the list for cost to repair or replace deficient highway bridges, with a total cost of repair of $1.07 billion. To put this into perspective, Wyoming, the tenth largest state in the country, ranked fiftieth on the list with a total cost of repair of $104 million [36].

Of Rhode Island's 6,401 miles of public road, 70% of roads are considered to be in poor or mediocre condition. Driving on these roads in need of repair with poor or deficient pavement cost Rhode Island motorists $350 million a year in extra vehicle repairs and operating costs, equivalent to $467 per motorist, or $143 more than the nation's average motorist in 2013. Just four years later this number increased to $810 in vehicle repairs and operating costs per year per Rhode Island motorist, or $277 more than the nation's average motorist in 2017 [37].

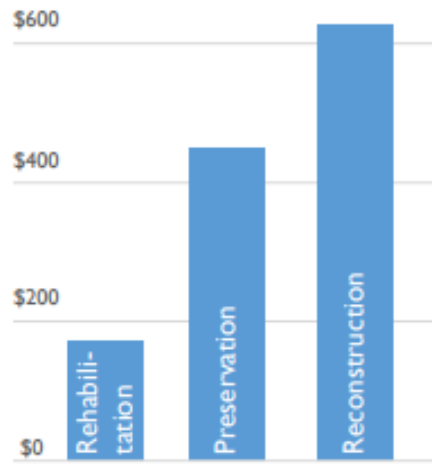
With so many of America's roadways and bridge decks in critical condition, immediate measures must be taken to either maintain new and good condition or to improve old and poor condition. It must be kept in mind that smaller and more continuous maintenance efforts in the present are much less expensive than larger and more complicated rehabilitation or replacement efforts in the future. Due to large deterioration contributors such as age and vehicle volume and use, bridge deck surfaces and roadway pavements have been negatively affected and are thus in poor condition. This poor condition pavement and surface severely affects not only each individual motorist, but also the nation as a whole.

1.6 State of Rhode Island Transportation Improvement Program

In December 2015, the Fixing America's Surface Transportation (FAST) Act, authorizing Federal highway, highway safety, transit and rail programs for 5 years from federal fiscal year (FFY) 2016 through 2020 was signed into law. Regulations require states to develop plans that specifically address how they will improve and sustain the conditions of roads and bridges. A goal of having no more than 10 percent of a state's bridge deck in poor condition was specified [47]. Currently 24.9% of Rhode Island's bridges are structurally deficient and in poor condition, which ranks Rhode Island last in the nation in overall bridge condition.

To meet federal standards, the State of Rhode Island Transportation Improvement Program (STIP) was adopted recently in September 2016 by the Rhode Island Department of Administration Division of Planning. Though the program includes maintenance for bridges, pavement, and traffic safety, the largest investment in the STIP is the Bridge Capital program. In order to stabilize Rhode Island's bridge condition, bridge maintenance is imperative. One of the largest shifts that has been occurring statewide is the migration of transportation infrastructure planning to an asset management based system of planning, which increases the emphasis on preservation and maintenance to keep assets in good condition, avoiding more expensive long term costs[47]. The Bridge Capital Program was developed using an asset management approach to identify and develop a structured sequence of preservation, repair, rehabilitation, and replacement actions that will achieve and sustain a desired state of good repair at a minimum practicable cost [47].

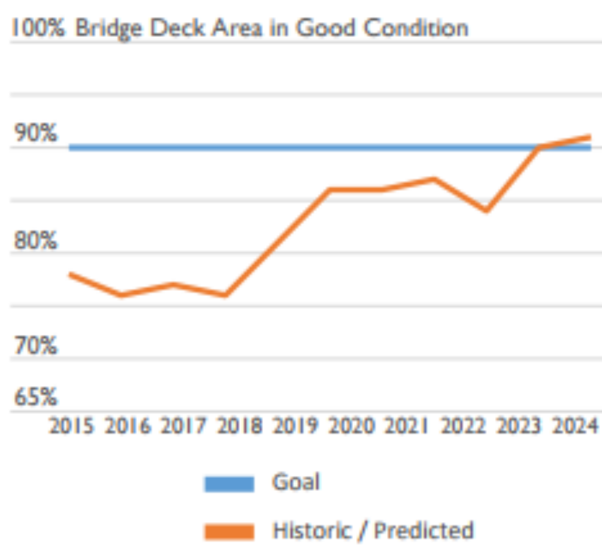
The STIP proposes a "surge" of bridge construction improvements in the first five years of the program, both for bridge reconstruction and preservation. Because bridge replacement is six times more expensive than bridge preservation, by investing more in bridge preservation efforts up front, the state can arrest the downward trend of bridge deterioration more cost-effectively [47].



Source: Rhode Island Department of Administration Division of Planning

Figure 1.6.1: Bridge Repair Costs per Square Foot

Once the STIP is implemented, the percent of structurally deficient bridges in Rhode Island would be reduced from 22% in 2014 to 10%, and the state can achieve the federal minimum standard of 90% bridge sufficiency by 2025 [47].



Source: Rhode Island Department of Administration Division of Planning

Figure 1.6.2: Rhode Island Bridge Deck Condition Goals

As a result of the improved bridge conditions, it is estimated that the state can save over $20 million in bridge inspection and emergency bridge repairs over the timeframe of the STIP. An asset management approach to maintenance means that every dollar invested today can save $3 in costs in future years [47].

While the STIP is established to improve existing bridge condition, it is important to also improve upon the evaluation of bridge deterioration. With so many structurally deficient Rhode Island bridges, implementing advanced technologies during routine bridge deck inspection can aid in better assessing both surface and subsurface deterioration, to better evaluate overall condition. Using non-destructive evaluation (NDE) techniques, a fuller picture of bridge deterioration can be assessed to aid in determining what management strategies should be implemented and when, for preservation purposes.

CHAPTER 2

BRIDGE DECK CONDITION ASSESSMENT METHODS

2.1 Simplistic NDE Methods

Though visible surface deficiencies such as cracks and potholes are reported during routine concrete bridge deck inspection, subsurface deterioration of reinforcement bar or concrete cover deficiency within a bridge deck is not accessible without more in-depth inspection. To make a more thorough assessment of concrete bridge deck condition, basic testing can be performed to determine areas of delamination within a bridge deck, rather than just on the surface. Hammer sounding and chain dragging are simplistic testing methods commonly used to assess and manage deterioration in concrete bridge decks.

Both methods are categorized as crude vibrational modal tests, and are often used to aid visual inspection of concrete structures. These testing methods are commonly used to specifically detect moderate to severe delamination in concrete bridge decks [15]. To perform the test, an operator drags chains or strikes a hammer on the deck, and listens to the resulting sound. The objective of these methods is to detect regions of a bridge deck where the sound from dragging the chains or hitting with a hammer changes from a clear ringing sound to a more muted and hollow sound. A clear ringing sound indicates a sound deck free from significant delamination, whereas a more muted and hollow sound indicates moderate to severe delamination. The hollow sound is a result of the flexural oscillations of the delaminated section of the deck, creating a drum-like effect that is within the audible range of the human ear [15].

Chain dragging is a quick method of testing used for determining the general location of moderately to severely delaminated areas of a concrete bridge deck. The speed of the chain drag varies with the level of deterioration of the deck, and the experience of the operator.

Source: United States Department of Transportation/Federal Highway Administration

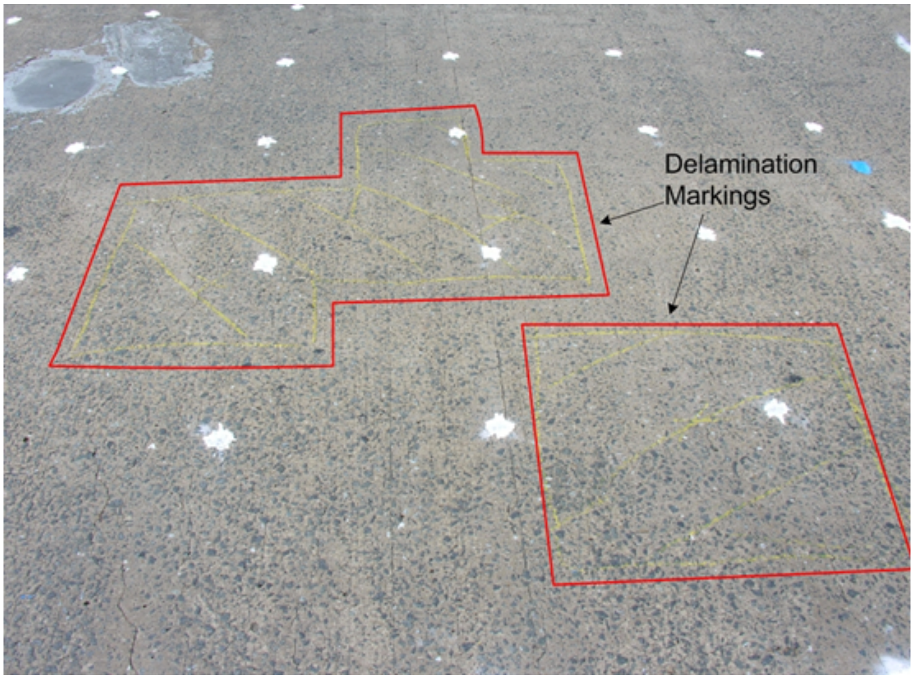
Figure 2.1.1: Chain Drag Testing

Hammer sounding is a slower method of testing, more appropriate for smaller areas. Hammer sounding can be used in conjunction with chain dragging in order to better define the size and extent of deterioration.

Source: United States Department of Transportation/Federal Highway Administration

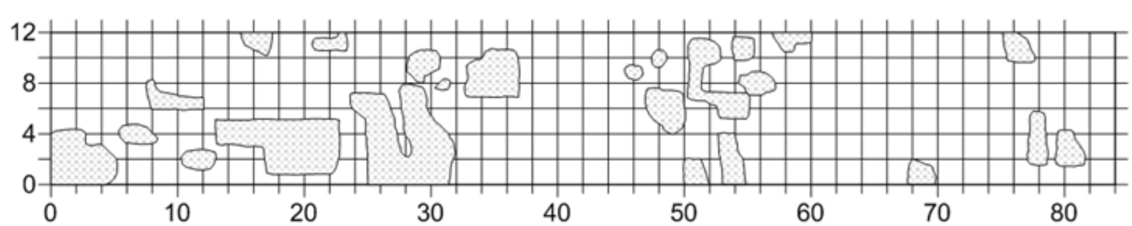
Figure 2.1.2: Hammer Sounding

Upon conducting chain drag testing, general areas of moderate to severe deterioration within a concrete bridge deck can be determined. Using hammer sounding, these areas of deterioration can be more accurately defined. Once accurately defined, the areas of deterioration can be physically marked on the bridge deck using semi-permanent chalk or spray paint, and from the markings, a computer generated deterioration map can be created. From the deterioration map, areas of delamination can be more accurately monitored and managed.



Source: United States Department of Transportation/Federal Highway Administration

Figure 2.1.3: Delamination Marking



Source: United States Department of Transportation/Federal Highway Administration

Figure 2.1.4: Deterioration Mapping

Hammer sounding and chain dragging are non-destructive testing methods commonly used to assess deterioration in concrete bridge decks, because they are quick and simplistic methods that do not require extensive training. With a skilled technician, these methods are cost-effective and capable of identifying areas of moderate to severe delamination within a concrete bridge deck. With these methods, deterioration that may be hidden to the naked eye upon visual inspection can be proactively detected, and deterioration maps can be easily generated to aid bridge management.

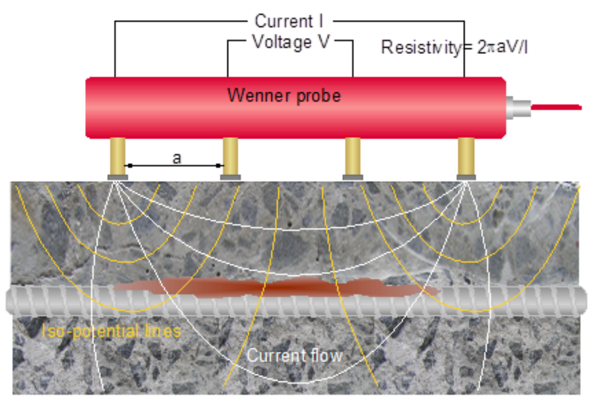
Though there are advantages to using hammer sounding and chain dragging, there are also limitations. These methods are labor intensive, and can only be performed when traffic noise is minimal. In addition, these methods are only capable of detecting moderate to severe deterioration, rather than the early onset of delamination. Because marked areas of deterioration rely on the meticulous ear of the operating technician, the results are highly subjective, and can vary from one technician to the next [15]. Therefore, traditional bridge deck inspection methods like hammer sounding and chain dragging are incapable of objectively detecting the early onset of deterioration, and are consequently less effective for both the assessment and management of subsurface deterioration in concrete bridge decks.

2.2 Advanced NDE Methods

The deterioration of bridge decks is commonly assessed and managed through visual deck surface inspection and through the use of simplistic subsurface methods such as chain dragging and hammer testing. Though cost-effective, these approaches only subjectively estimate deterioration, and may only detect deterioration after it is too late in its moderate to severe stages. Using advanced technologies allows for the proactive detection of deterioration, often before it is visible to the naked eye and early enough to make a difference before substantial deterioration occurs. Advanced technologies include electrical resistivity (ER), half-cell potential (HCP), impact-echo (IE), ground penetrating radar (GPR), and surface roughness testing.

Electrical Resistivity (ER)

Electrical resistivity (ER) or its reciprocal, electrical conductivity, is an intrinsic property that quantifies the ability of a given material to oppose or conduct electric current. In other words, ER testing can be performed to determine reinforced concrete's susceptibility to corrosion [13].With the presence of corrosive substances such as water, chlorides, and salts, damaged and cracked areas of a bridge deck will form preferential paths for fluid and ion flow, creating a corrosive environment. This leads to higher moisture and chloride concentrations and higher concrete electrical conductivity, manifesting as a lower electrical resistivity. The lower the electrical resistivity of the concrete, the higher the current passing between anodic and cathodic areas of the reinforcement steel will be [13].



Source: United States Department of Transportation/Federal Highway Administration

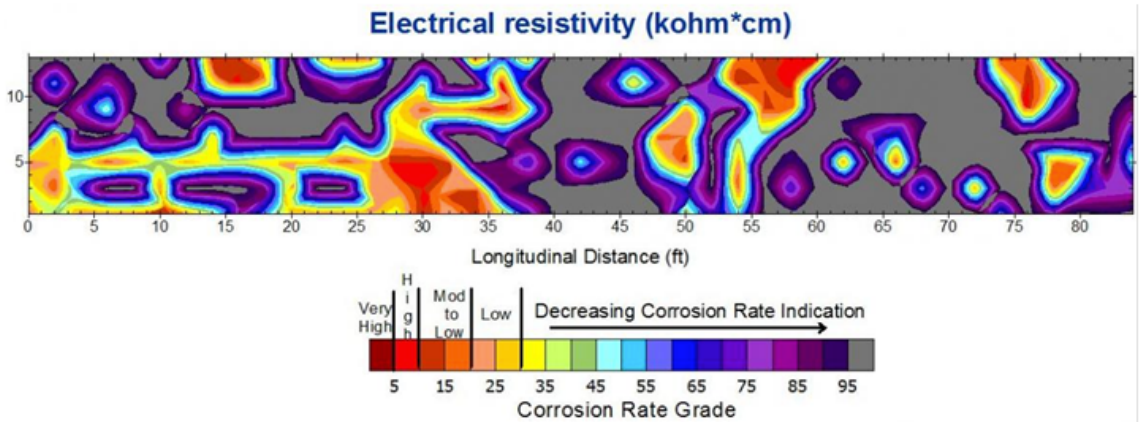
Figure 2.2.1: ER Testing Principle

Surface electrical resistivity of a steel-reinforced concrete element (typically, the cover of a steel-reinforced concrete slab or deck) is an indicator of concrete corrosive environment. To conduct electrical resistivity tests, the voltage and current are measured at the surface of the object under investigation using a certain layout of electrodes. A current is applied between the two outer electrodes, and the potential is measured across the two inner ones [13].

Source: United States Department of Transportation/Federal Highway Administration

Figure 2.2.2: ER Testing

The resistivity is then calculated. Areas with low resistivity are indicative of corrosion, whereas areas with high resistivity are free from corrosion [13]. The X and Y coordinates of the test section can be plotted against the electrical resistivity measurements to create a corrosion map.



Source: United States Department of Transportation/Federal Highway Administration

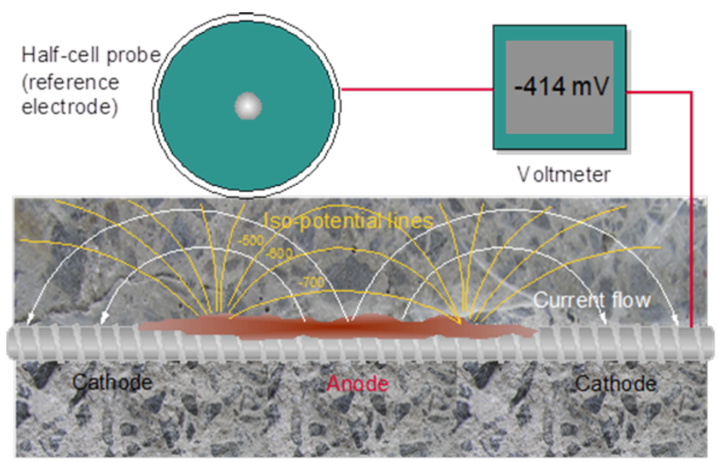
Figure 2.2.3: ER Corrosion Map

ER testing is a cost-effective, repeatable method of testing to assess corrosion that does not require a high level of expertise for data collecting or data processing. It can however be time consuming and labor intensive, and the data can be significantly impacted by a number of environmental parameters such as moisture, salt content, and porosity. Unlike half-cell potential testing, ER testing does not directly measure corrosion.

Half – Cell Potential (HCP)

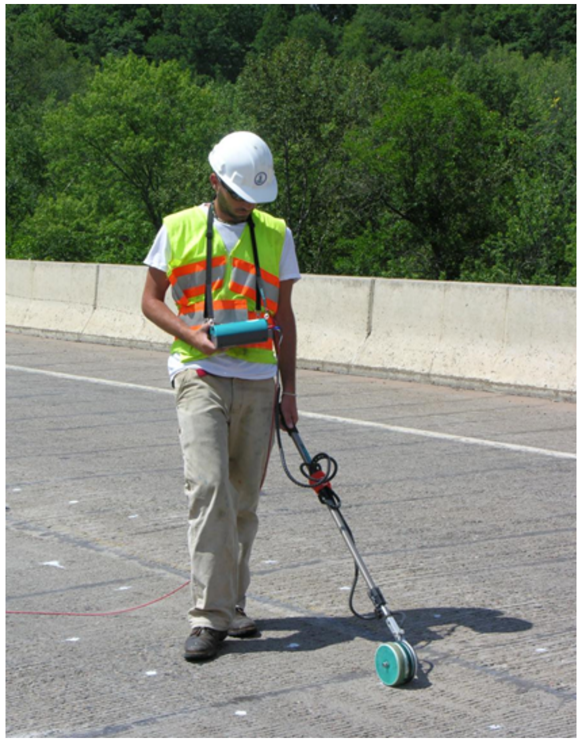
Half-cell potential (HCP) testing is an electrochemical method of testing that can be performed to identify corrosion activity of steel reinforcement in reinforced concrete structures. HCP measurement is based on the coexistence of anodic and cathodic half-cells, or corroding and non-corroding areas on reinforcement bar. The measurement is calculated as the difference in potential, or voltage, across the steel-concrete interfaces[16]. The potential difference between a standard portable half-cell and the reinforcing steel of a concrete element is measured. When the reference electrode is moved along a line or grid on the surface of a member, the spatial distribution of corrosion potential can be mapped. Any change in the potential between the reference electrode and the steel-concrete interface can be attributed to, among other things, the corrosion activity at the surface of the steel [16].

Corrosion of steel in concrete is similar to the electrochemical mechanism of corrosion of a metal in an electrolyte. This implies that separate anodic and cathodic processes take place simultaneously on the same metal surface. At the corroding side (the anode), iron is dissolved and oxidized to iron ions, leaving electrons in the steel. At the cathodic side of the reaction, oxygen is reduced and hydroxyl ions are produced. The potential of the generated electrical field is measured by a reference electrode. The reference electrode is connected to the positive end of a voltmeter and steel reinforcement to the negative one. The reference electrode is usually galvanically coupled to the concrete surface using a wet sponge [16].



Source: United States Department of Transportation/Federal Highway Administration

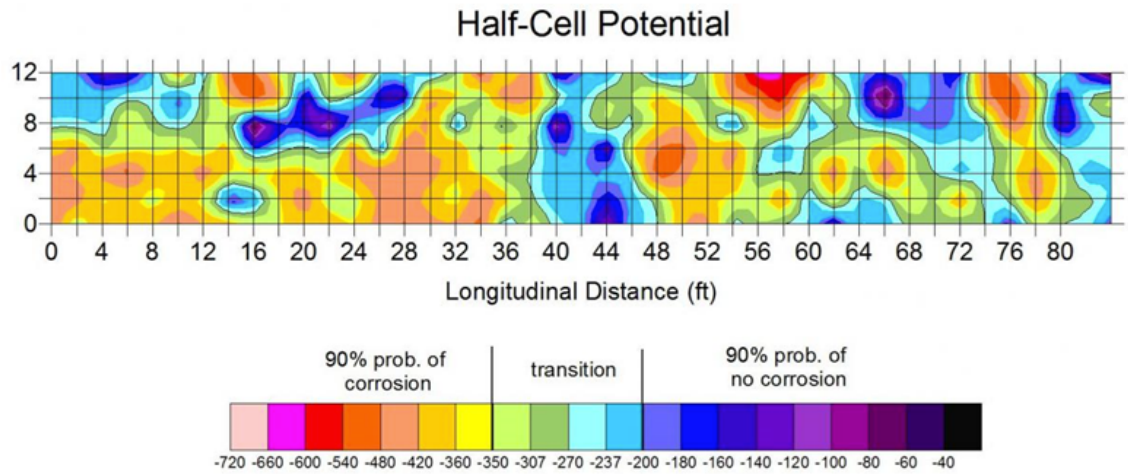
Figure 2.2.4: HCP Measurement Principle



Source: United States Department of Transportation/Federal Highway Administration

Figure 2.2.5: HCP Testing Using a Rolling Probe

Once HCP data are collected, X and Y coordinates can be plotted against measured voltage to produce a map showing areas of very high likelihood for active corrosion, very low likelihood for active corrosion, or a transition zone that spans the measurements in between in accordance with American Society for Testing and Materials (ASTM) standards [16].



Source: United States Department of Transportation/Federal Highway Administration

Figure 2.2.6: HCP Corrosion Mapping

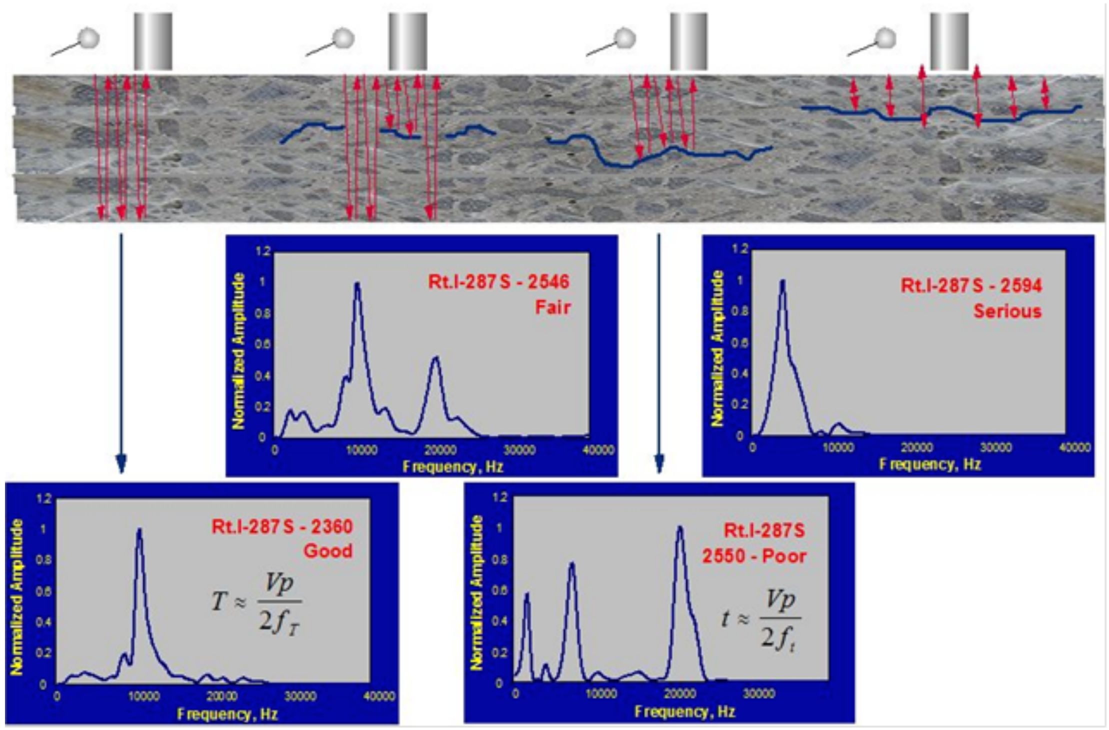
Though HCP testing is a quick and easy way to assess corrosion of reinforcing steel within a concrete bridge deck, it is not without limitations. HCP testing can result in erroneous and unreliable measurements due to isolating layers such as asphalt, coating, and paint on the deck surface or coated rebar, and also if the concrete is wet, dense, or polymer-modified. In addition, HCP testing cannot be performed if electrical continuity does not exist in the element being evaluated.

Impact Echo (IE)

Impact echo (IE) testing is a seismic or stress-wave based method of testing used to detect defects in concrete, primarily delamination [25]. IE equipment consists of an impactor and a sensor, used to detect and characterize wave reflectors in concrete elements. IE testing works by first distributing an impact to the ground surface that generates propagation waves within the tested material. External boundaries, as well as any areas with internal defects, will reflect waves with a difference in acoustic impedance. When reflected waves, or echoes, return to the surface, displacements are produced and the transient response time of the material is measured with the sensor. The amplitude spectrum obtained from the fast Fourier transform analysis of the time signal will show dominant peaks at certain frequencies, which can be interpreted to assess the deck condition [25].

Source: United States Department of Transportation/Federal Highway Administration

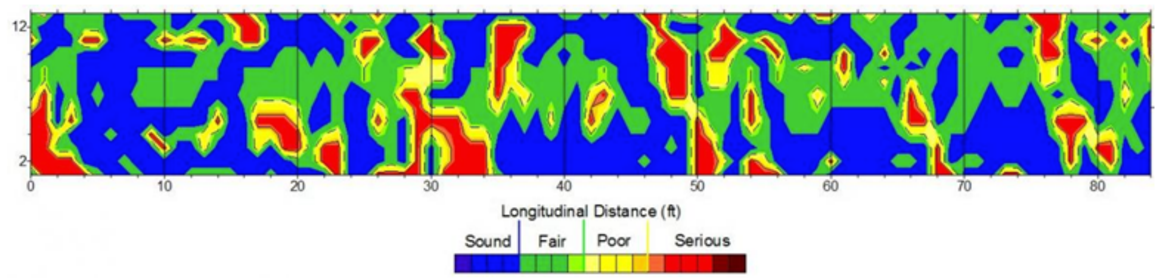
Figure 2.2.7: IE Testing Using Manual Probe



Source: United States Department of Transportation/Federal Highway Administration

Figure 2.2.8: IE Testing Principle

The response of returned echoes is dependent on the severity of the delamination. A sound deck (good condition) will have a distinctive peak in the response corresponding to the full depth of the deck. An initial delamination (fair condition) is identified through the presence of two distinct peaks, indicating energy partitioning from two dominant wave propagation patterns, the first peak corresponding to reflections from the bottom of the deck and the second one to reflections from the delamination. Progressed delamination (poor condition) is characterized by a single peak at a frequency corresponding to a reflector depth that is shallower than the deck thickness, indicating that little or no energy is being propagated towards the bottom of the deck. Finally, in the very severe case of a wide or shallow delamination (serious condition), the dominant response of the deck to an impact is characterized by the low-frequency response of flexural mode oscillations of the upper delaminated portion of the element [25]. Upon categorizing the IE measurements, the X and Y coordinates can be plotted against the severity in order to create a delamination condition map.



Source: United States Department of Transportation/Federal Highway Administration

Figure 2.2.9: IE Delamination Condition Map

IE testing is advantageous because it is capable of detecting delamination at very early stages, with reliable and repeatable results when conducted properly by an experienced operator. Limitations with IE arise as testing with traditional single probe equipment is extremely slow, and requires a dense grid to accurately define the boundaries of delaminated areas. In addition, the collection, processing, and interpretation of IE data requires significant training and expertise.

<u>Ground Penetrating Radar (GPR)</u>

Ground Penetrating Radar (GPR) is a geophysical, non-destructive method of testing that can be used in a variety of applications to determine subsurface layers, objects, and voids. Among these many applications, GPR has been largely used for subsurface discovery, mapping, and imaging for forensic, military, geology, and inspection purposes. GPR is an accepted advanced technology that essentially provides an X-ray of the desired test section. This subsurface imagery is used to see what can be hidden from the surface and to the naked eye, including soil stratification, underground utilities, and voids. For these types of applications, dangerous target locations can be marked so that they can be avoided. Specifically for applications such as concrete bridge decks, GPR can be used to identify concrete cover thickness and areas in which the cover is non-compliant, as well as reinforcement bar depth, spacing, and condition.

In order for subsurface imaging to produce accurate findings, the test application must first be evaluated in terms of material and desired depth penetration. GPR subsurface depth penetration is mainly dependent upon two conditions: the survey material type and the frequency of the antenna used. Lower conductivity materials allow for increased depth penetration. Lower frequency antennas are capable of penetrating these deeper depths, but with decreased target detection and resolution. Contrastingly, higher conductivity materials that tend to absorb GPR signals allow only for shallower depth penetration. Though higher frequency antennas are capable of penetrating only shallower depths, target detection and resolution is increased. Therefore dependent on the survey material, desired depth penetration, and target size, choice of antenna is one of the most important factors for testing [17].

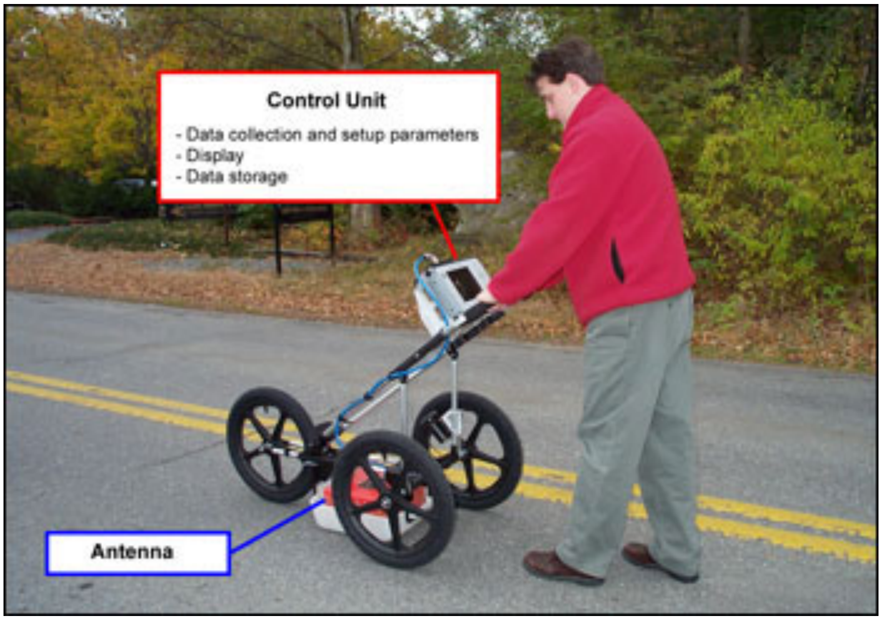
Table 2.2.1: Appropriate Antennas Based on Application and Depth Range

Appropriate Application	Primary Antenna Choice	Secondary Antenna Choice	Depth Range (Approximate)
Structural Concrete, Roadways, Bridge Decks	2600 MHz	1600 MHz	0-0.3 m (0-1.0 ft)
Structural Concrete, Roadways, Bridge Decks	1600 MHz	1000 MHz	0-0.45 m (0-1.5 ft)
Structural Concrete, Roadways, Bridge Decks	1000 MHz	900 MHz	0-0.6 m (0-2.0 ft)
Concrete, Shallow Soils, Archaeology	900 MHz	400 MHz	0-1 m (0-3 ft)
Shallow Geology, Utilities, UST's, Archaeology	400 MHz	270 MHz	0-4 m (0-12 ft)
Geology, Environmental, Utility, Archaeology	270 MHz	200 MHz	0-5.5 m (0-18 ft)
Geology, Environmental, Utility, Archaeology	200 MHz	100 MHz	0-9 m (0-30 ft)
Geologic Profiling	100 MHz	MLF (16-80 MHz)	0-30 m (0-90 ft)
Geologic Profiling	MLF (16-80 MHz)	None	Greater than 30 m (90 ft)

Source: Global GPR Services, Inc.

For determining the subsurface conditions of a concrete bridge deck, a higher frequency antenna of 1.6 GHz is used for the higher resolution detection of shallowly located reinforcement bar.

GPR equipment works by first triggering a pulse of radar energy from the control unit to the antenna. The antenna receives the electrical pulse produced by the control unit, amplifies it and transmits it into the ground or other medium at a particular frequency.



Source: Geophysical Survey Systems, Inc.

Figure 2.2.10: Ground Coupled GPR GSSI Equipment

Source: Geophysical Survey Systems, Inc.

Figure 2.2.11: GPR GSSI Control Unit

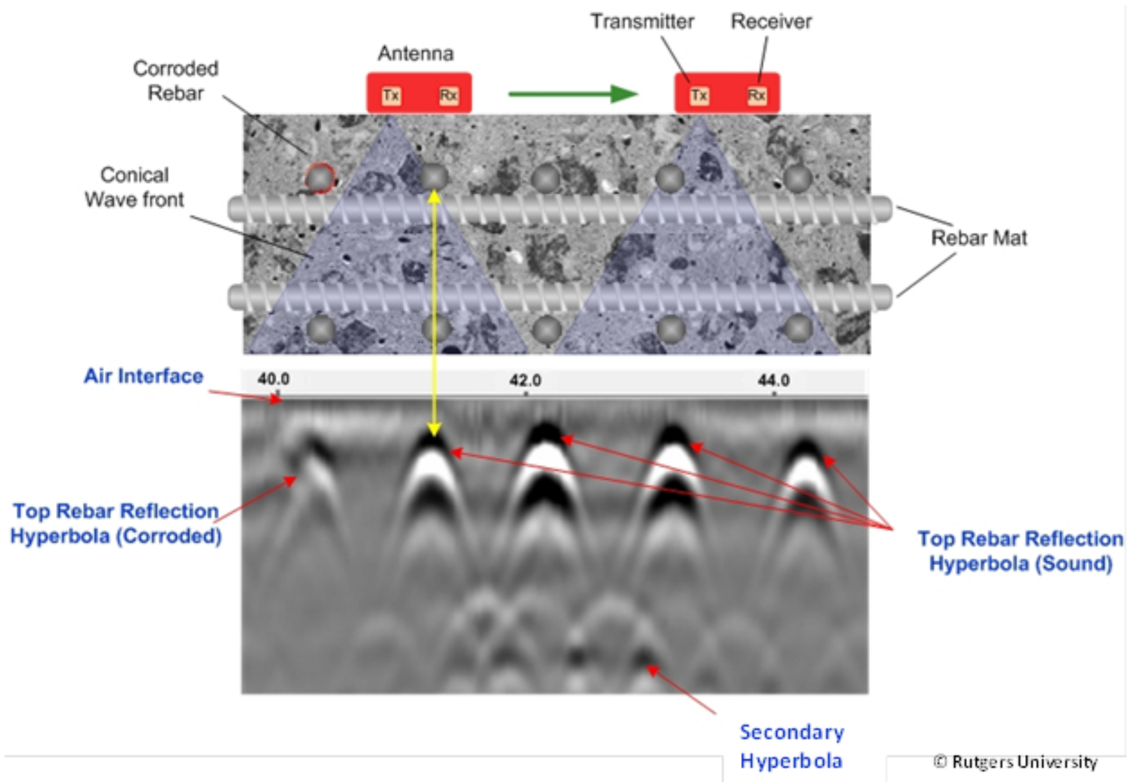
After sending the tiny pulse of energy into a material, the strength and time required for the return of any reflected signal is recorded. Reflections are produced whenever the energy pulse enters into a material with different electrical conduction properties from the material it left. The strength, or amplitude, of the reflection is determined by the contrast in the conductivities of the two materials [18].

Table 2.2.2: Typical Dielectric Values for Various Pavement Materials

Material	Dielectric	
	Typical Range	Signal Attenuation
HMAC	5 - 7	Low
HMAC - stripped	< 4	Low
AC w/light weight aggregate	3 - 4	Low
AC – wet (surface or voids)	> 8	Mod
Open Graded Friction Course (OGFC) (Plant Mix Seal [PMS], Permeable Friction Course [PFC])	3.5 - 4.5	Low
OGFC w/light weight aggregate	2 - 4	Low
Microsurfacing	3.5 - 4.5	Low
Flex Base – dry	< 8	Low
Flex Base at optimum moisture content (OMC)	8 - 12	Low
Flex Base – saturated	> 16	High
Cement Treated Base	7 - 9	Low
Clay – wet	> 20	High
Concrete – old	8	Mod
Concrete – new	10 - 20	High
Air	1	Low
Water	81	High
Ice	3	Low

Source: Geophysical Survey Systems, Inc.

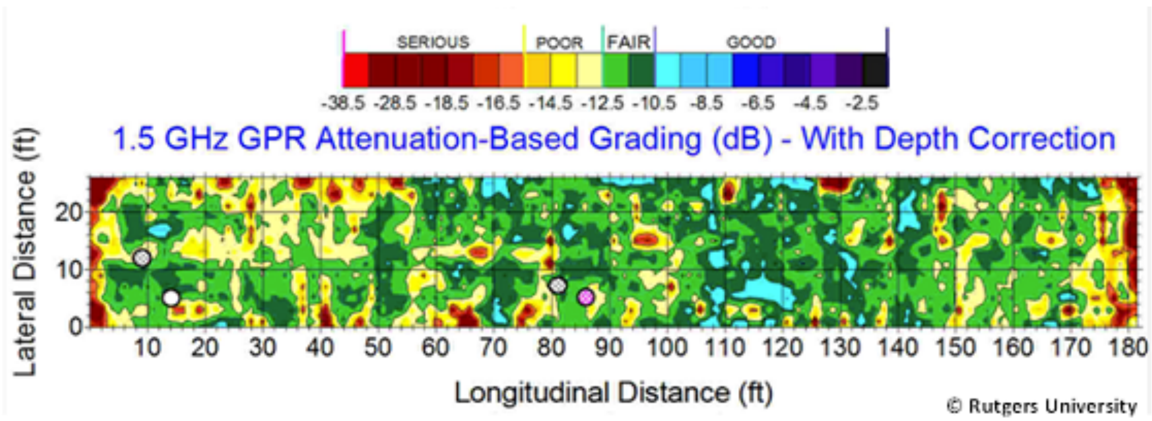
When testing concrete bridge decks, the deterioration of the concrete cover or reinforcement bar can be determined based on the change, or attenuation, in amplitude strength. A larger change or difference in return signal amplitude from the least deteriorated point is indicative of deterioration. A series of pulse reflections over a single area make up what is called a scan [18].



Source: Rutgers University

Figure 2.2.12: GPR Testing Principle

From the antenna, radar energy pulses are emitted in a cone-like shape. Because of this cone shape, the two-way travel time for a signal is longest when approaching or moving away from a target, and shortest when directly over the target. That is, as the antenna is moved over a target, the distance between them decreases until the antenna is directly over the target, and increases as the antenna is moved away. It is for this reason that a single target will appear in the data as a hyperbola. The target is actually at the peak amplitude of the positive wavelet [18]. Obtained through field-testing, the scans can be transferred from the GPR equipment to a computer to be used in accordance with the specialized software RADAN. Processing the data in RADAN allows for the determination of the exact location, depth, and amplitude of each target or reinforcement bar. With this information concrete cover and reinforcement bar deterioration within a bridge deck can be evaluated. Defined by the difference in amplitude strength of returned signals from the least deteriorated point, bridge deck deterioration can be mapped.



Source: Rutgers University

Figure 2.2.13: GPR Deterioration Map

GPR testing is a rapid, reliable, and repeatable non-destructive method of testing that correlates well with electrical resistivity to describe corrosive environments, and well with other non-destructive evaluation (NDE) methods when the defects are severe. GPR testing however, can be negatively influenced by extremely cold conditions, saturated conditions, and de-icing agents. In addition, GPR testing requires advanced expertise and training for data collection, processing, and interpretation.

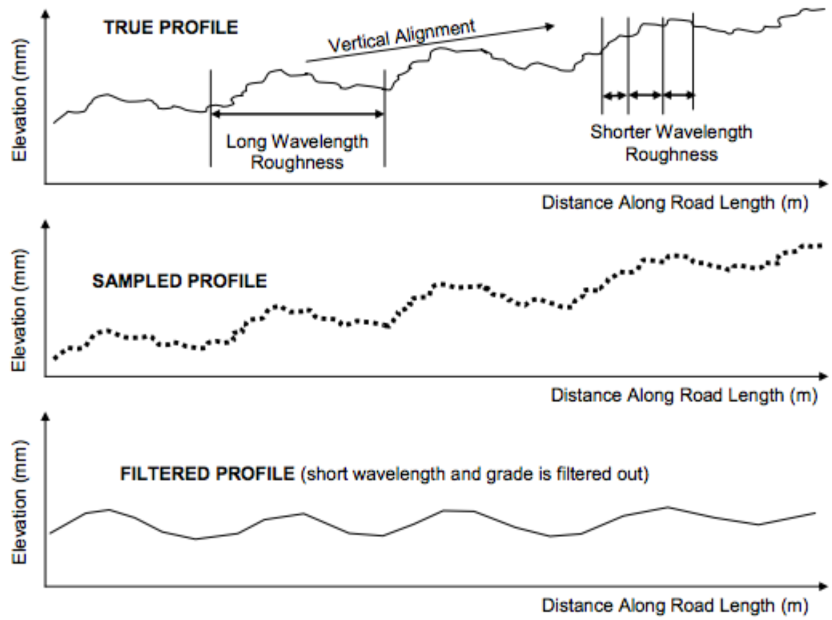
Table 2.2.3: Non-Destructive Evaluation (NDE) Techniques for the Assessment of Deterioration in Concrete Bridge Decks

Assessment Method	Use	Advantages	Limitations
Visual Inspection	• Routine bridge inspection • Detect visible deterioration	• Simple • Cost-effective • Immediate, no data processing necessary	• Detects surface defects • Inconsistent reporting (subjective) • Time-consuming • Difficult to quantify
Hammer Sounding & Chain Dragging (HSCD)	• Aid visual inspection • Detect subsurface delamination in concrete structures	• Simple, no extensive training • Cost-effective • Immediate, no data processing necessary	• Detects only moderate to severe delamination • Labor intensive • Only performed when traffic noise is minimal • Dependent on ear of skilled technician, highly subjective
Electrical Resistivity (ER)	• Determine reinforced concrete's susceptibility to corrosion	• Cost-effective • Repeatable • Does not require a high level of expertise	• Time consuming and labor intensive • Data can be significantly impacted by moisture, salt content, and porosity • Does not directly measure corrosion
Half-Cell Potential (HCP)	• Electrochemical method of testing • Identify corrosion activity of steel reinforcement in reinforced concrete structures	• Simple, no extensive training • Quick	• Can result in erroneous/unreliable measurements due to isolating layers such as asphalt, coating, and paint on the deck surface or coated rebar, and also if the concrete is wet, dense, or polymer-modified • Cannot be performed if electrical continuity does not exist in the evaluated element
Impact Echo (IE)	• Seismic or stress-wave based method of testing • Detect and characterize wave reflectors in concrete elements	• Capable of detecting delamination at very early stages • Reliable and repeatable results	• Dependent on being conducted properly by an experienced operator • Time consuming and labor intensive • Collection, processing, and interpretation of data requires significant training and expertise
Ground Penetrating Radar (GPR)	• Geophysical method of testing • Detect subsurface layers, objects, and voids • Subsurface imaging	• Rapid • Repeatable • Correlates well with other NDE methods	• Negatively influenced by extremely cold conditions, saturated conditions, de-icing agents • Requires advanced expertise and training for data collection, processing, and interpretation

2.3 Surface Roughness

Described in simplest form surface roughness, otherwise known as road roughness or ride quality, is a term used to quantify the level of comfort or discomfort a motorist feels when traveling a roadway, or a bridge deck. Data and information gathered during surface roughness testing can be used to quantify bridge deck surface condition and quality. Both the condition and quality of a bridge deck can be affected by common imperfections including rutting, cracking, potholes, local failures, etc. Each of these imperfections causes changes in surface elevation along the road profile; therefore measuring the road profile is the most direct method of quantifying these surface elevation deviations.

From a test section, a true profile can be generated to display the variations in surface elevations over distance. The true profile can then be subdivided into a number of sinusoidal curves of varying wavelength, of which only wavelengths pertinent to surface roughness can be extracted. A filtered profile that excludes grade variation and waves irrelevant to surface roughness can then be used to determine a roughness parameter representative of surface condition.



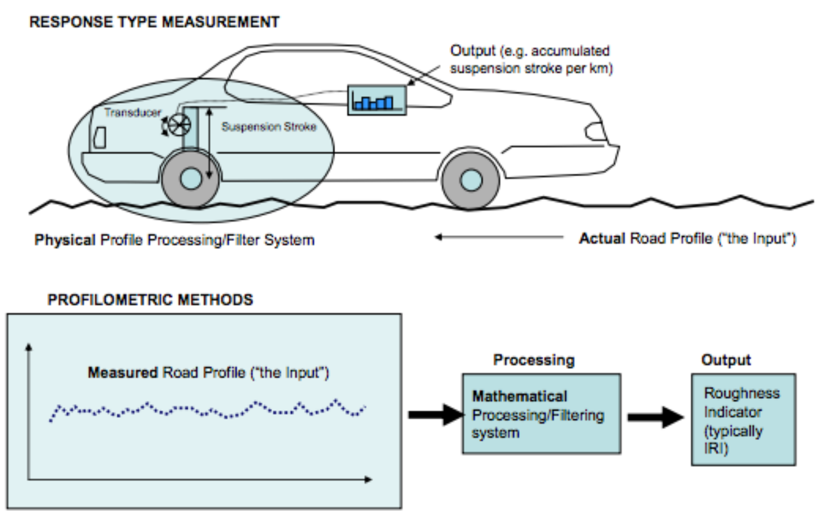
Source: South Africa Committee of Transportation Officials

Figure 2.3.1: Profile Measurement Concepts

The measurement of surface roughness can be classified into two basic types: response type measurement and profilometric type measurement. Response type measurement is used to directly measure the response of a measurement vehicle to a traveled section of road. In this type of measurement, the road profile is never actually measured, but rather the vehicle's response to the profile is measured and quantified. When using response type measurement, a parameter known as the Average Rectified Slope (ARS) can be determined as an output from the vertical movement of the vehicle. Rather than describing the actual elevation contours of the road over distance, as the road profile does, the ARS parameter describes the up and down movement of the suspension, normalized by the distance covered [24].

Profilometric type measurement involves the measurement of the road profile, after which the profile is filtered, to determine a parameter called the International Roughness

Index (IRI). The filtering and processing of the road profile is designed to simulate the response of a standard vehicle to the measured profile. Profilometric type measurement is generally preferred in comparison to response type measurement because it provides more consistent data without variable factors such as vehicle type and suspension system properties. This measurement approach however, requires significantly more expensive equipment and in-depth understanding and monitoring of the measured data than does response type measurement.

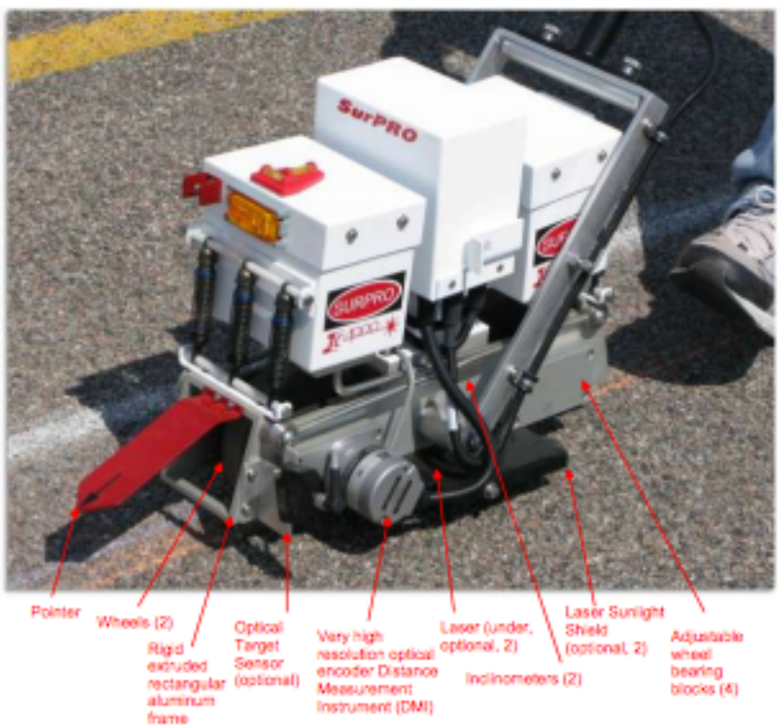


Source: South Africa Committee of Transportation Officials

Figure 2.3.2: Surface Roughness Measurement Types

Upon comparison, determining the IRI values using profilometric type measurement is generally more preferable than determining the ARS values using response type measurement. Response type measurement is dependent on the damping and stiffness properties of the measurement vehicle, which can vary over time. In turn, these varying properties fail to provide consistent ARS data. In contrast, a key advantage of using IRI data is that the IRI parameter is calculated using a computer algorithm that will naturally remain constant over time, allowing for IRI data to be reproducible [24].

Today, van-mounted response type measurement devices are most often used to measure surface roughness because they are capable of collecting the data quickly. Though data can be collected at a faster pace, the accuracy of collected data is decreased, and ARS values only partially quantify the actual road profile roughness. Rather, with decreased speed, using a profilometric type measurement like the walk-behind surface profiler SurPRO allows for more accurate collection of data, and produces IRI values that better represent true roughness.



Source: International Cybernetics Corporation, Inc.

Figure 2.3.3: SurPRO Equipment

Using the data collected from surface roughness testing and ProVAL software, deck surface condition maps can be generated using MATLAB to display areas of visible surface deterioration more accurately than simple visual inspections.

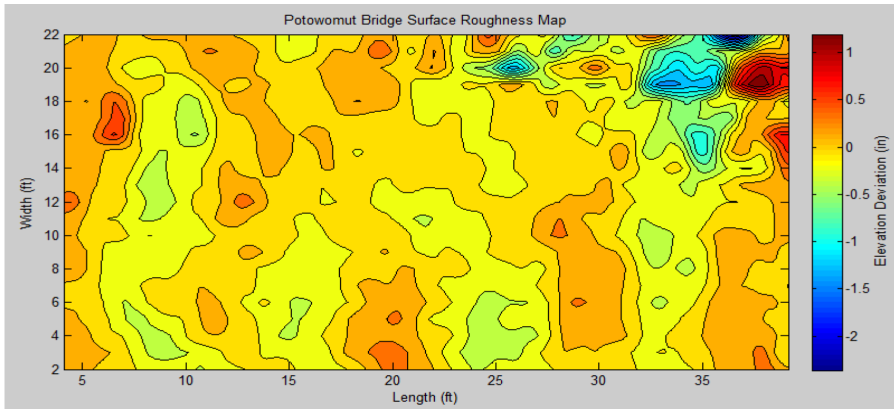
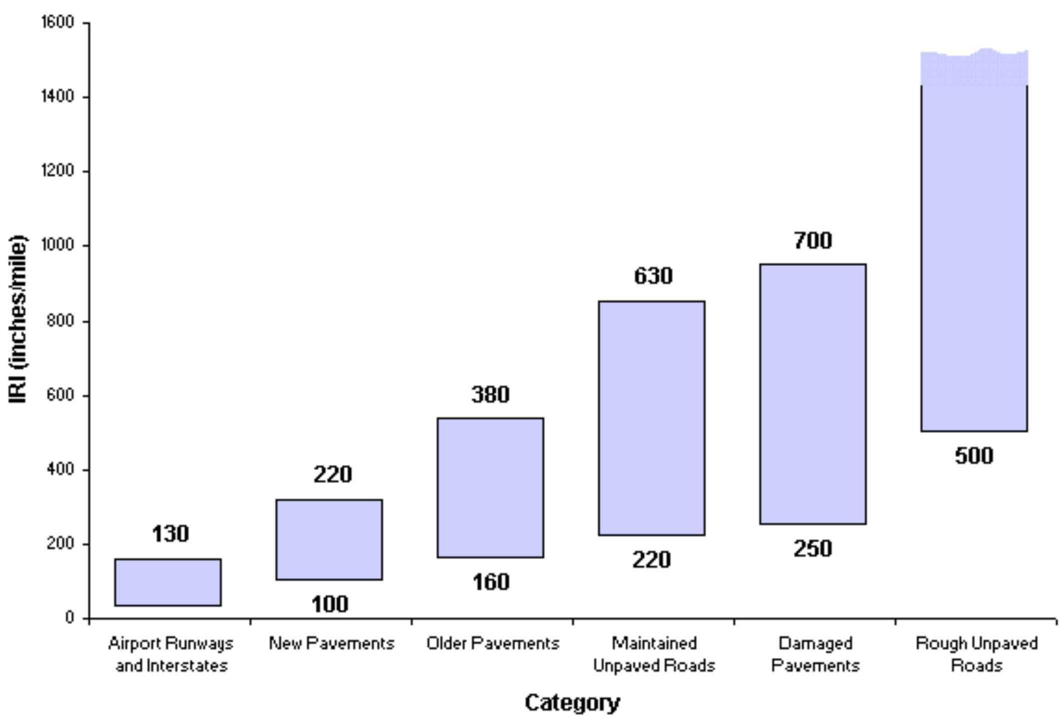


Figure 2.3.4: Surface Roughness Map

Surface roughness mapping is capable of describing bridge deck deficiency in more detail than can traditional visual inspection methods, providing more precise deck deficiency quantity, severity, and location information. In addition to a visual representation of bridge deck surface deficiency, the International Roughness Index (IRI) can be calculated so that overall surface deficiency can be quantified, then compared either to other bridge decks, or to previous condition to determine the extent and rate of deck surface deterioration.

The IRI is a roughness parameter that simulates the displacement of one wheel of a typical passenger car, and is often referred to as the "quarter car model". In the IRI calculation, the measured profile is processed using a mathematical transform that filters and cumulates the wavelengths throughout a profile. The transform was developed and calibrated in a manner that ensures that the IRI output is closely correlated with road user perception of roughness and tire load dynamics, which have significant impacts on vehicle control and safety [24]. Upon filtering the raw roughness data collected during testing, the IRI algorithm eliminates all wavelength components that do not contribute to roughness experienced by road users, and highlights the roughness elements that have the greatest impact of perceived roughness for road users. Thus in essence, the IRI is calculated through a mathematical simulation of the physical response of a typical vehicle to a road profile [24].



Source: American Society for Testing and Materials

Figure 2.3.5: Surface Roughness IRI Scale

Bridge deck surface deficiency can be quantified with the determination of the IRI value, where a larger IRI value is representative of pavement or bridge deck surfaces in poorer condition. Very low IRI values relate to airport runways and superhighways, and very high IRI values relate to rough or unpaved roads or surfaces, with new pavements/surfaces, older pavement/surfaces, and damaged pavements/surface in between. A higher IRI value is indicative of an increased amount of surface imperfections typical with damaged pavements, including depressions, erosion, and potholes. This IRI value is useful because it allows for the quantification of overall deck deficiency, so that the condition of a bridge deck over time can be better monitored and managed.

CHAPTER 3

FIELD TESTING OF IN-SERVICE BRIDGES

3.1 Study Objective

The deterioration of bridge decks is commonly assessed and managed through visual deck surface inspection and through the use of simplistic subsurface methods such as hammer sounding and chain dragging. Though cost-effective, these approaches are subjective, and only capable of detecting deterioration in its moderate to severe stages. To assess deterioration within a bridge deck more thoroughly, the use of advanced technologies can be incorporated into routine bridge deck inspection to view what may be hidden from the naked eye and missed using traditional assessment methods.

Through the use of advanced technologies, bridge deck condition can be more accurately assessed and therefore more accurately reported following inspection. When accurately reported, the rate at which a bridge deck is deteriorating can be determined, and therefore smaller and more cost-effective management strategies can be implemented before substantial deterioration occurs or continues. With new and improved methods for assessing concrete bridge deck deterioration, both surface and subsurface, maintaining good bridge condition preserves the structural integrity, as well as the comfort, cost, and safety of the public, while extending lifespan.

The objective of this study is to analyze surface roughness and ground penetrating radar data collected from field testing, to demonstrate that a more thorough assessment of surface and subsurface deterioration in Rhode Island concrete bridge decks can be obtained through the use of advanced technologies. Three bridge decks, visually in good, moderate, and poor condition, are initially tested in 2015 to generate surface and subsurface deterioration maps then tested a second time two years later (the length of time of a typical routine bridge inspection) in 2017, to study the effects of time on subsurface deterioration. Both initial and secondary findings are compared to reported bridge inspection deck conditions to assess accuracy in reported bridge deck condition. The subsurface conditions of the original test will be compared to those of the secondary test, to determine change in condition over the two-year time period. It is important to evaluate the change in subsurface condition over time, to best determine what management strategies should be implemented and when, for preservation purposes.

3.2 Research Methodology

Using surface roughness to map visible surface deck deficiencies and ground penetrating radar to map invisible subsurface deck deficiencies, three Rhode Island bridges of varying visual deck condition were tested. Major Nathanael Greene Bridge in Coventry, Rhode Island, Ramp BB Bridge in North Kingstown, Rhode Island, and Potowomut Bridge in Warwick, Rhode Island were chosen as test bridges in visually good, moderate, and poor condition, respectively.

The bridges were tested initially for surface and subsurface deficiencies using surface roughness and ground penetrating radar equipment, then tested a second time two years later to determine the change in subsurface conditions. The objective of ground penetrating radar testing two years later is to study the change in subsurface conditions that occurs within a concrete bridge deck that may not be identified during routine bridge inspection. Secondary findings are compared to initial findings to determine the change in subsurface condition. All findings are compared to reported deck condition to determine if surface and subsurface deterioration, as well as any changes in subsurface condition, are accurately reported.

3.3 Bridge Information

The Rhode Island Department of Transportation (RIDOT) provided access to three bridge decks for testing using GPR and a surface profiler. All testing was conducted between June - August 2015. The bridges were of varying types and represented exposed concrete and asphalt overlay decks. Bridges were generally selected based on access and impact to traffic. Because lane closures were required during testing, RIDOT generally selected low volume bridges. A general description for each bridge is provided below. Bridge locations are mapped in Figure 3.3.1.

Source: Google Maps

Figure 3.3.1: Location of Rhode Island Bridges used for Testing

In 2015 and in 2017 additional information for each bridge relating to its condition was obtained from an online site (www.uglybridges.com) that makes use of public NBI information. The latest reported deck condition ratings were obtained from the NBI database.

Table 3.3.1 General Description of Bridges Tested

Bridge Name	Location	Structure Type	Deck	NBI Deck Condition Rating (as of date)	NBI Deck Condition Rating (as of date)	NBI Deck Condition Rating (as of date)
Major Nathanael Greene	Laurel Avenue, Coventry RI	Multi-beam or girder steel bridge	Concrete CIP w/ monolithic concrete wearing surface	9 (2012)	7 (2015)	7 (2017)
Ramp BB	RI 403 Ramp BB over Ramp EE/ W. Davisville Rd./RR, North Kingstown, RI	Prestressed concrete multi-girder bridge	Concrete CIP w/ monolithic concrete wearing surface	7 (2013)	7 (2015)	7 (2017)
Potowomut	Old Forge Road, Warwick RI	Multi-beam prestressed box girder bridge	Concrete CIP w/ bituminous wearing surface	7 (2013)	7 (2015)	7 (2017)

In 2017, the latest information for each bridge relating to its condition was obtained from routine bridge inspection reports provided by the Rhode Island Department of Transportation (RIDOT). This information is presented in the tables below.

Table 3.3.2 Major Nathanael Greene Bridge Description

Bridge (Structure Number):	Laurel Avenue (Major Nathanael Greene Bridge) over the Pawtuxet River, Coventry 000000000003970	**Lat/Long:**	+41.695574, -71.546925
Purpose:	Carries highway and pedestrian walkway over waterway		
Structure:	Multi-beam or girder steel bridge	**Length:**	91.9 ft
Deck:	Concrete CIP w/ monolithic concrete wearing surface	**Width:**	32.0 ft curb-to-curb
ADT/Truck Traffic:	4,850 (10%)	**Year Built:**	1900, 2012 (reconstructed)
Condition Rating (out of 9) as of October 2016:		**Appraisal Rating (out of 9) as of October 2016:**	
Deck:	Good [7]	Structural:	Equal to present desirable criteria [8]
Superstructure:	Good [8]	Deck geometry:	Meets minimum tolerable limits to be left in place as is [4]
Substructure:	Good [8]	Underclearances:	Not available
Capacity:		Water adequacy:	Equal to present desirable criteria [8]
Design Load:	MS18/HS20	Roadway alignment:	Equal to present desirable criteria [8]
Operating Rating	66.1 tons	**Sufficiency Rating:**	97.0
Inventory:	50.7 tons	**Evaluation:**	Not Deficient

Source: Rhode Island Department of Transportation Bridge Inspection Report

Table 3.3.3 Ramp BB Bridge Description

Bridge (Structure Number):	RI 403 Ramp BB over Ramp EE/ W. Davisville RD/RR 000000000010110	**Lat/Log:**	+41.60417, -71.44833
Purpose:	Carries highway over highway and railroad		
Structure:	Prestressed concrete stringer/multi-beam or girder bridge	**Length:**	133.5 ft total (128.9 ft largest span length)
Deck:	Concrete CIP with monolithic concrete wearing surface placed concurrently with structural deck	**Width:**	32.0 ft from curb to curb
ADT/Truck Traffic:	2,650 (3%)	**Year Built:**	2002
Condition Rating (out of 9) as of June 2017:		**Appraisal Rating (out of 9) as of June 2017:**	
Deck:	Good [7]	Structural:	Better than present minimum criteria [7]
Superstructure:	Good [7]	Deck geometry:	Above tolerable [5]
Substructure:	Good [7]	Underclearances:	Equal to present minimum criteria [6]
Capacity:		Water adequacy:	Not available
Design Load:	MS22.5/HS25	Roadway alignment:	Above tolerable [5]
Operating Rating:	67.7 tons	**Sufficiency Rating:**	97.8%
Inventory:	44.7 tons	**Evaluation:**	Not Deficient

Source: Rhode Island Department of Transportation Bridge Inspection Report

Table 3.3.4 Potowomut Bridge Description

Bridge (Structure Number):	Old Forge Road (Potowomut) Bridge over Hunt River, Warwick 000000000009910	**Lat/Long:**	+41.629837,-71.453139
Purpose:	Carries highway and pedestrian walkway over waterway		
Structure:	Multi-beam or girder prestressed concrete bridge	**Length:**	42.0 ft span (49.9 total)
Deck:	Concrete CIP w/ bituminous wearing surface	**Width:**	21.98 ft curb-to-curb
ADT/Truck Traffic:	1,515/ (10%)	**Year Built:**	2002

Condition Rating (out of 9) as of December 2015:		**Appraisal Rating (out of 9) as of December 2015:**	
Deck:	Satisfactory[6]	Structural:	Equal to present minimum criteria [6]
Superstructure:	Good [7]	Deck geometry:	Intolerable - correct [3]
Substructure:	Good [7]	Underclearances:	Not available
Capacity:		Water adequacy:	Superior to present desirable criteria [9]
Design Load:	MS18/HS20	Roadway alignment:	Above minimum criteria [7]
Operating Rating	44.0 tons	**Sufficiency Rating:**	74.0%
Inventory:	33.0 tons	**Evaluation:**	Not Deficient

Source: Rhode Island Department of Transportation Bridge Inspection Report

Three ratings are listed in the tables above, namely the condition, appraisal and sufficiency rating. Condition ratings are used to describe the existing, in-place bridge as compared with the as-built condition. They act as the major source of information on the status of the bridge and reflect the deterioration or damage of structural members.

Although field inspections are completed for each element, condition ratings provide an overall characterization of the general condition of the three main areas of a bridge – deck, superstructure, and substructure. A scale of 0 to 9 is used to represent failed condition (closed bridge) and excellent condition, respectively [27].

An appraisal rating is used to evaluate a bridge in relation to the level of service which it provides on the highway system of which it is a part. It allows the in-service bridge to be compared to a newly built bridge using current standards. It too uses a 0 to 9 rating scale representing a closed bridge to one that is superior to present desirable criteria, respectively [27].

The sufficiency rating is based on a formula aimed to represent the bridge sufficiency to remain in service. This rating is represented as a percentage in which 100% represents a perfectly sufficient bridge and 0% represents an entirely insufficient bridge. The formula uses information relating to the structural adequacy and safety, serviceability and functional obsolescence, essentiality for public use, and special reductions for detour length, certain bridge types, and lack of traffic safety features [27].

Code	Description
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION - no problems noted.
7	GOOD CONDITION - some minor problems.
6	SATISFACTORY CONDITION - structural elements show some minor deterioration.
5	FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
4	POOR CONDITION - advanced section loss, deterioration, spalling or scour.
3	SERIOUS CONDITION - loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	"IMMINENT" FAILURE CONDITION - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION - out of service - beyond corrective action.

Figure 3.3.2: FHWA Condition Rating Descriptions [27]

Code	Description
N	Not applicable
9	Superior to present desirable criteria
8	Equal to present desirable criteria
7	Better than present minimum criteria
6	Equal to present minimum criteria
5	Somewhat better than minimum adequacy to tolerate being left in place as is
4	Meets minimum tolerable limits to be left in place as is
3	Basically intolerable requiring high priority of corrective action
2	Basically intolerable requiring high priority of replacement
1	This value of rating code not used
0	Bridge closed

Figure 3.3.3: FHWA Appraisal Rating Descriptions [27]

3.4 Field Testing Setup

The field testing process was very similar for each bridge. Testing occurred on days where it had not rained that day or the day before, so that the collected dataset was not affected by moisture. In general, a 1-ft by 2-ft grid is marked along the entire length of the bridge with 1-ft in the transverse direction and 2-ft along the longitudinal or travel direction. The 2-ft longitudinal marks are meant to ensure a straight travel path with the testing equipment and the denser 1-ft transverse markings provide more opportunities for data collection.

The first longitudinal line generally extended about 1 to 2 feet from the curb and each subsequent line was marked every foot until either the other curb was reached if testing the entire bridge deck, or near the center lane marking if testing only half of the bridge. Some distance was kept between the curb or center lane marking and the longitudinal line used for testing in order to avoid traffic traveling in an adjacent lane and to provide space for the equipment.

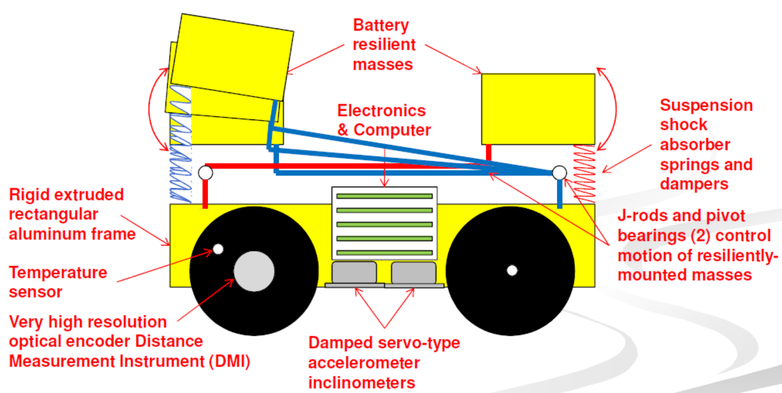
Figure 3.4.1: Field Testing Setup (a) Lane Closure and Traffic Control (b) Transverse Grid Markings (c) Longitudinal Grid Markings (d) Data Collection with the Surface Profiler (e) Data Collection with GPR

<u>Surface Profiler</u>

The surface profiler used in this research is the SurPRO 4000 developed by International Cybernetics Corporation (ICC). The SurPRO 4000 is a rolling or walking multipurpose surface profiling instrument used to measure surface elevation profiles. These profiles can then be used to calculate various indices including the International Roughness Index (IRI), Ride Number (RN), and profilograph profile index (PI).

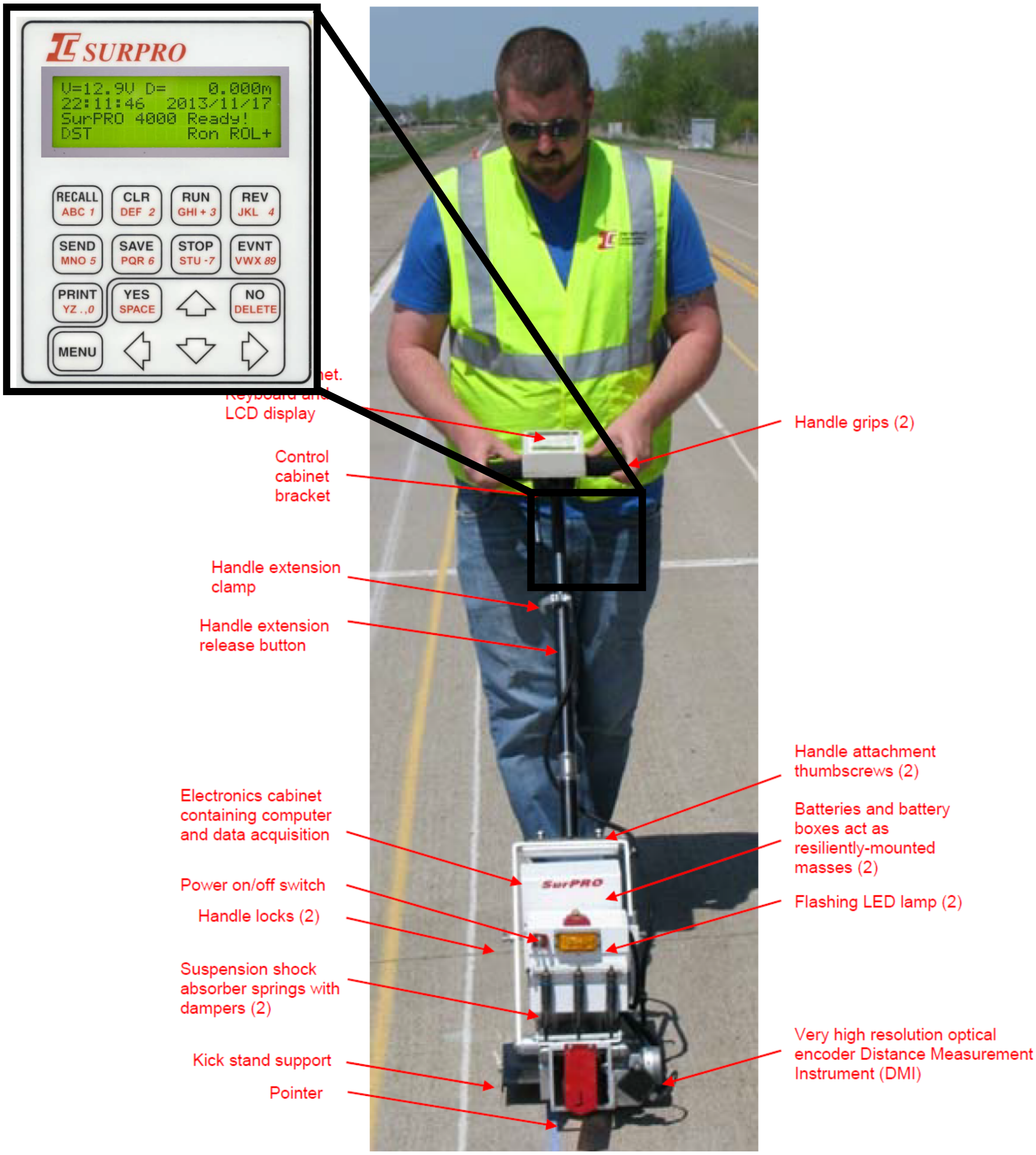
The SurPRO is equipped with two inclinometers, a longitudinally and a transversely-aligned high-accuracy, high-resolution force-balance accelerometer, that measure the orientation of the frame, a high resolution optical encoder distance measuring instrument (DMI), and a temperature sensor as shown in Figure 3.4.2. Other components of the equipment are shown in Figure 3.4.3.

Prior to testing, the profiler usually undergoes two calibrations; a distance calibration to calibrate the DMI and an elevation calibration to calibrate the longitudinal inclinometer (i.e. closed loop). The latter is completed by performing a closed loop profile. Once the equipment is calibrated, the profiler is pushed along each longitudinal grid line along the length of the bridge at a steady pace of about 1-2 MPH. At the end of the bridge, the profile is saved and the profiler is brought back to the beginning of the bridge and positioned along the next longitudinal grid line. The process continues until the bridge deck has been profiled. More detailed instructions and system settings used during testing are provided in Appendix A.



Source: International Cybernetics Corporation, Inc.

Figure 3.4.2: Sensors and Key Components of the SurPRO 4000



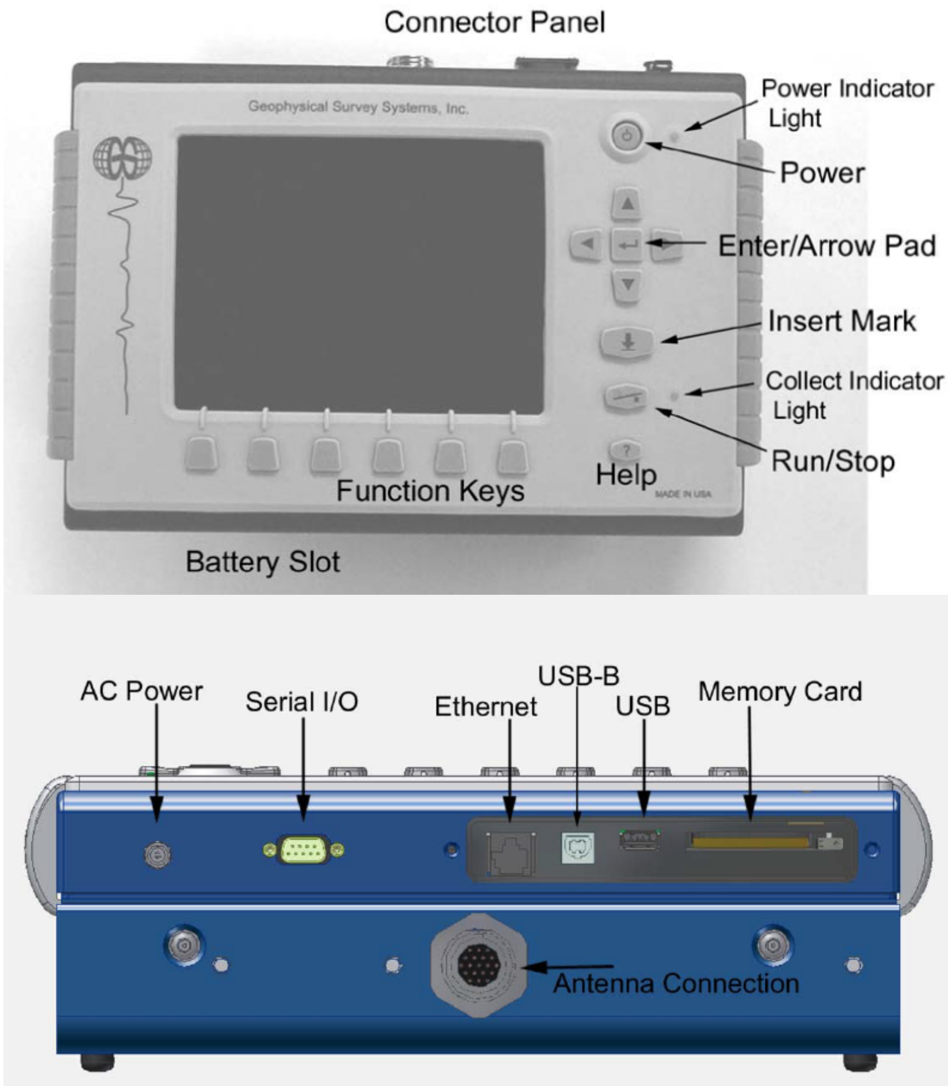
Source: International Cybernetics Corporation, Inc.

Figure 3.4.3: Components of the SurPRO 4000

Ground Penetrating Radar

The GPR system used in this research is from Geophysical Survey Systems, Inc. (GSSI) and has three main components as shown in Figure 3.4.5. The SIR-3000 controller is a portable, single-channel GPR system with a display screen that allows data to be viewed in real time or in playback mode. The controller, shown in Figure 3.4.6, is connected to a distance measuring instrument (DMI) installed on the wheel and a 1.6 GHz center frequency ground-coupled antenna housed in a white bin that skims the roadway surface. All components are attached to a durable survey cart. Data are collected at a rate of 120 scans/ft (10 scans/in) over a range of 15 ns/scan.

Figure 3.4.5: Components of the GSSI GPR Equipment



Source: Geophysical Survey Systems, Inc.

Figure 3.4.6: Components of the SIR-3000 Controller (Top) Face of Controller (Bottom) Back of Controller

GPR data are collected in a similar manner as the surface profiler. Once the DMI has been calibrated, the survey cart is pushed along a longitudinal profile at a walking pace. The system is set to collect 10 samples/inch and will sound a beep if the operator walks too fast. Once the end of the bridge is reached along the first longitudinal line, the run is ended and the survey cart is brought back to the beginning of the bridge and positioned on the next longitudinal line. This process continues until the bridge deck is scanned. More detailed instructions and system settings used during testing are provided in Appendix A.

CHAPTER 4

EVALUATION OF IN-SERVICE BRIDGES

Raw data collected in the field using surface roughness and ground penetrating radar equipment were processed using an assortment of software programs, to evaluate surface and subsurface deterioration respectively. With deterioration maps, calculated quantification parameters such as the International Roughness Index (IRI), and statistical parameters such as mean and standard deviation of reflected return signal amplitude attenuation, the location and severity of both surface and subsurface deterioration can be better reported thus providing a clearer picture of overall bridge deterioration for better assessment.

4.1 Surface Roughness and Surface Mapping

Once surface profiles for the bridge deck had been collected as described in the previous chapter, the data were exported from SurPRO and analyzed using the software ProVAL [29]. ProVAL (Profile Viewing and AnaLysis) is an engineering software sponsored by the FHWA and the Long Term Pavement Performance Program (LTPP). It is used to view and analyze pavement profiles collected by a variety of profilers.

Once imported into ProVAL each raw profile was viewed, processed and analyzed. A raw profile is shown in Figure 4.1.1. When the raw profile is viewed here in its entirety, little detail is shown of the actual elevation deviations along the measured profile, and rather the change in overall elevation of a measured road profile is displayed. Upon analyzing the sinusoidal curves, it can be noted that not all wavelengths are of great importance in regards to roughness measurement. In fact, many vehicle suspension systems are designed to remove or dampen the effect of many of the wavelengths in a profile. Wavelengths that are very long typically relate to vertical alignment and slope, and wavelengths that are very short typically relate to surface texture. The wavelengths that have the greatest influence on user comfort are those between 1 and 30 meters. When a road profile is processed to compute roughness, the wavelengths outside of this critical range, as well as the grade of the road are typically filtered out [24]. A filtered profile is shown in Figure 4.1.2.

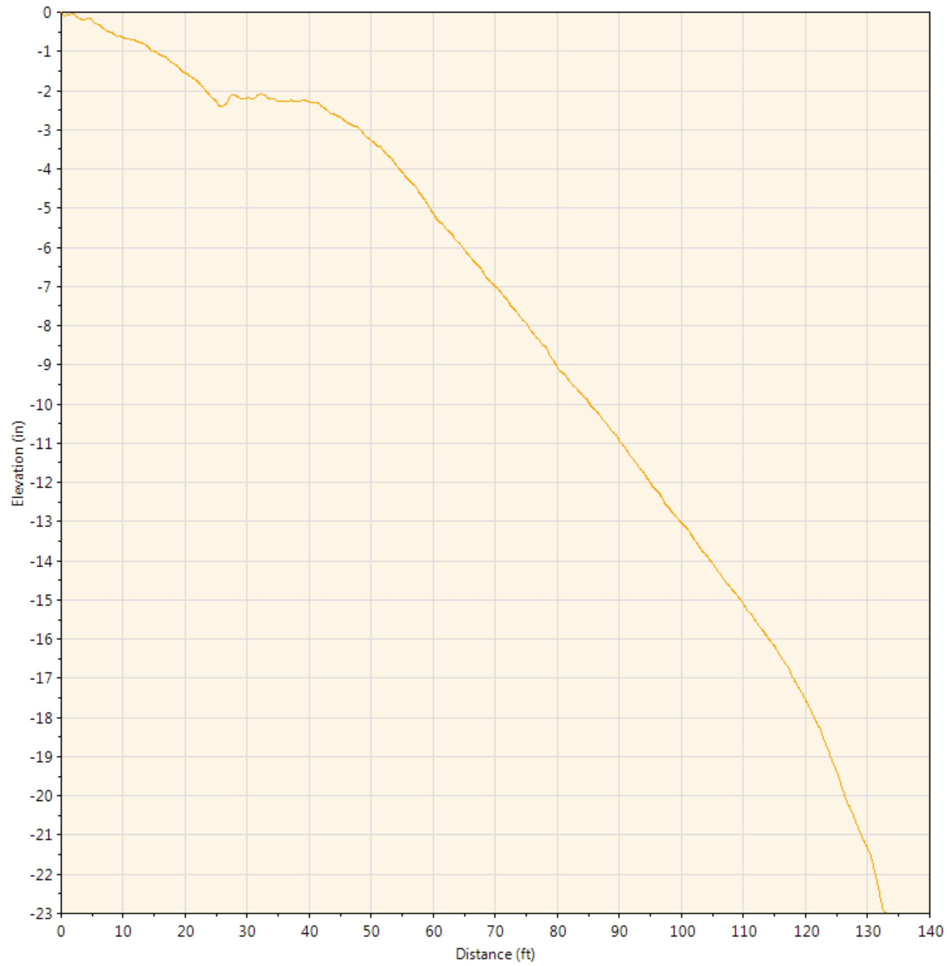


Figure 4.1.1: Raw Surface Profile from Ramp BB Bridge at 2 ft from Curb

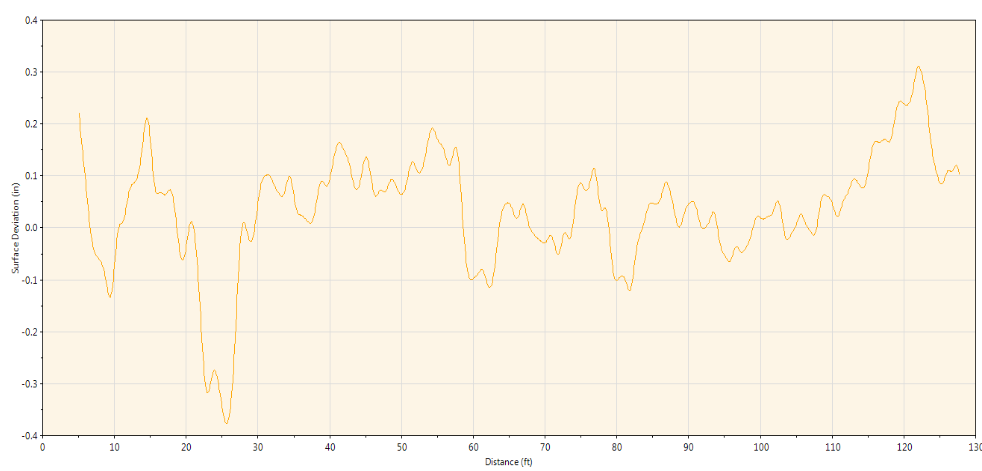
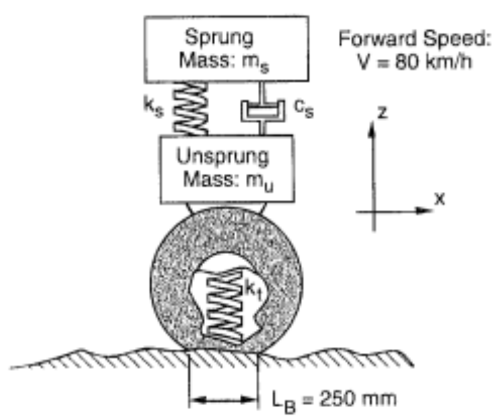


Figure 4.1.2: Filtered Surface Profile from Ramp BB Bridge at 2 ft from Curb

Once the desired profile is produced, free of irrelevant data and wavelengths, the "Ride Quality" analysis was performed for each individual profile. This analysis allows for the full, fixed interval, and continuous report of ride indexes including the Mean Roughness Index (MRI), the Ride Number (RN), the Half-Car Roughness Index (HRI), and most importantly the International Roughness Index (IRI).

The IRI is a roughness parameter that simulates the displacement of one wheel of a typical passenger car. It is characterized by specific processing algorithms: a moving average filter and the quarter-car model, which simulate the physical properties and displacement of a vehicle wheel and suspension system. Thus in essence, the IRI is calculated through a mathematical simulation of the physical response of a typical vehicle to a road profile [24]. A moving average filter is used to simulate the enveloping behavior of pneumatic tires on highway vehicles, and to reduce the sensitivity of the IRI algorithm to the sample interval. The quarter-car model includes the major dynamic effects, masses, springs, and dampers, which determine how roughness causes vibrations in a road [44].



Source: Sayers

Figure 4.1.3: IRI Variables

The quarter-car model is described by the four first-order ordinary differential equations that can be written in matrix form. The IRI is an accumulation of the simulated motion between the spring and unsprung masses in the quarter-car model, normalized by the length of the profile [44].

All information obtained from surface roughness data, including the IRI value and elevation deviations along length, was exported to Excel. The IRI values calculated for each of the profiles were averaged together to find an overall IRI for the entire test section of the bridge. The Excel dataset that included elevation deviations along the length of each measured profile, produced using ProVAL, was then imported to MATLAB [26] to generate surface roughness maps by interpolating between profiles across the width of the deck. Results are shown in Figure 4.1.4 through Figure 4.1.6 for each of the three bridges. Table 4.1.1 provides a summary of the surface roughness for all three bridges and includes the IRI values, maximum variation in surface deviations, and the deck condition rating from 2015 bridge inspections as reported by the NBI.

Table 4.1.1: Summary of Surface Condition of Bridges

Bridge (Date Tested)	**ADT (%Trucks)**	**IRI (in/mi) Pavement Condition**	**Max. Variation in Elevation (in)**	**NBI Deck Condition Rating (as of date)**
Major Nathanael Greene (06/17/15)	4,850 (10%)	142.80 new pavement	0.37	7 (2015)
Ramp BB (08/12/15)	2,650 (3%)	279.50 older pavement	0.81	7 (2015)
Potowomut (07/07/15)	1,515 (10%)	539.97 damaged pavement	3.55	7 (2015)

Using the IRI values, it is clear that Potowomut has the most severe surface roughness conditions but the index provides no information as to the location of such deterioration. By examining the surface roughness map, however, it becomes clear that the damage is localized to one corner of the bridge. This type of information is helpful for bridge deck management as it is able to quantify and locate the damage. It is interesting to note that all three bridges have a deck condition rating of 7 as reported by the NBI, although the IRI value of Potowomut is nearly double that of Ramp BB and four times that of Major Nathanael Greene, with maximum variation in elevation more than four times and nearly ten times, respectively.

It is important to note the elevation deviation scale on each of the surface roughness maps at first, as the scale is not universal for the three candidate bridges. In keeping the elevation deviation scale unique to each bridge deck, areas red in color will always represent the highest elevations, and areas blue in color will always represent the lowest elevations. Therefore the most deteriorated areas, specific to each bridge deck, can be identified. Though areas of deterioration, the elevation deviation scale must be referenced, because deterioration can describe anywhere from 0.17 to 1.19 inches or -0.2 to -2.36 inches in this particular study.

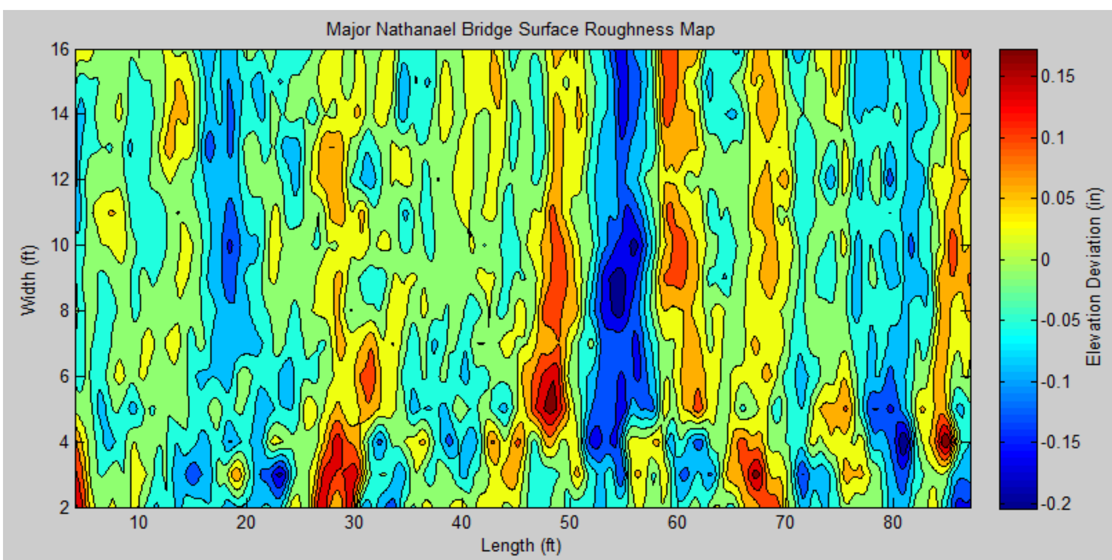


Figure 4.1.4: Major Nathanael Greene Bridge (Top) Deck Surface (Bottom) Surface Roughness Map

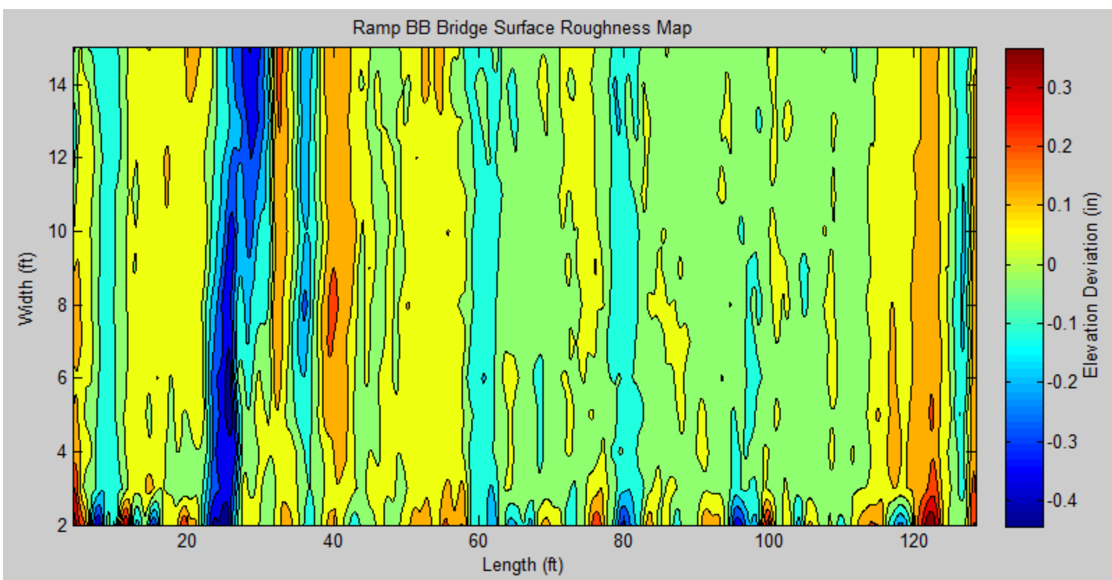


Figure 4.1.5: Ramp BB Bridge (Top) Deck Surface (Bottom) Surface Roughness Map

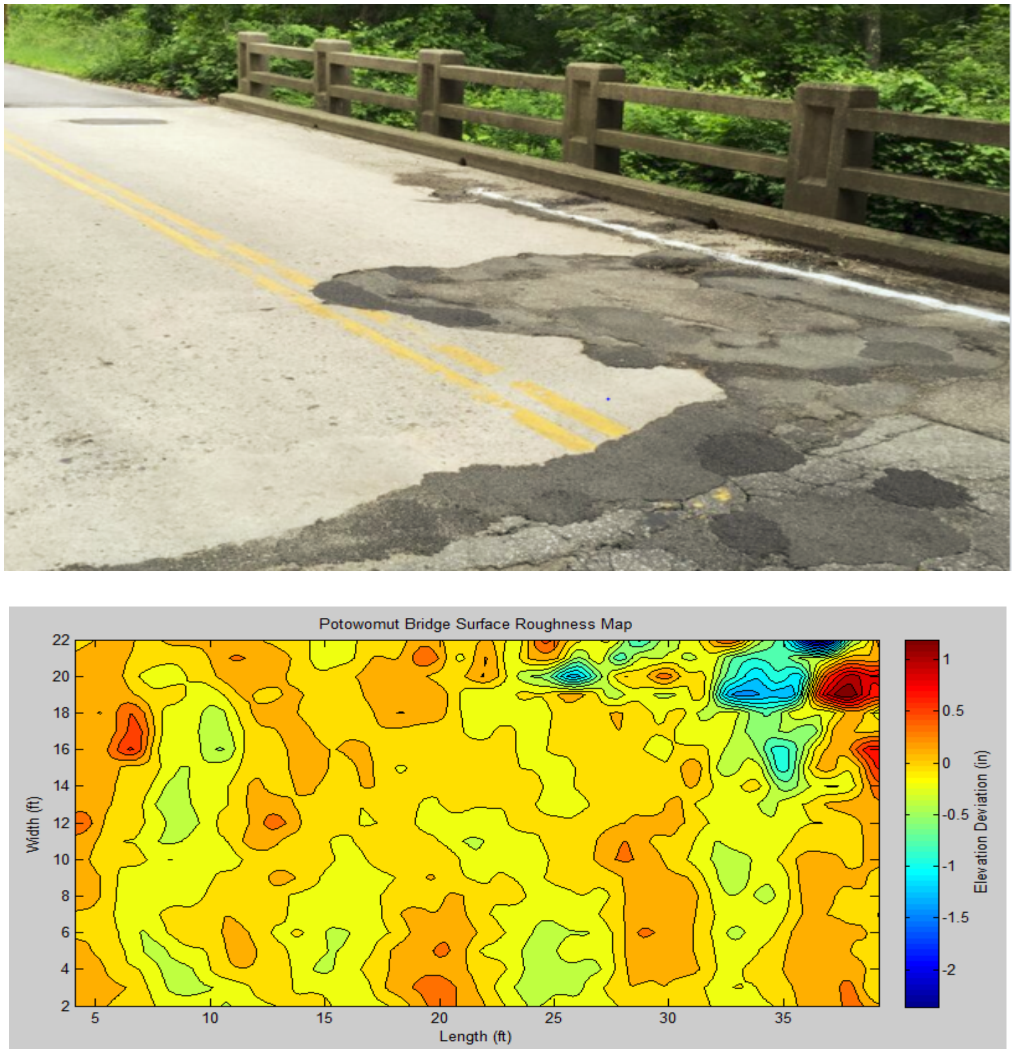


Figure 4.1.6: Potowomut Bridge (Top) Deck Surface (Bottom) Surface Roughness Map

Upon generating each of the surface roughness maps, it is determined that the Potowomut Bridge has the most significant surface deficiencies, with elevation deviation ranging from -2.36 inches to 1.19 inches. Though it is useful to visualize deck surface deficiency unique to each bridge deck, in order to meaningfully compare the surface condition of each of the three bridges to one another, the elevation deviation scale was made universal. This universal scale was made the worst-case scenario, that of the Potowomut

Bridge. The surface roughness maps for each of the three bridges with the universal scale are shown in Figure 4.1.7.

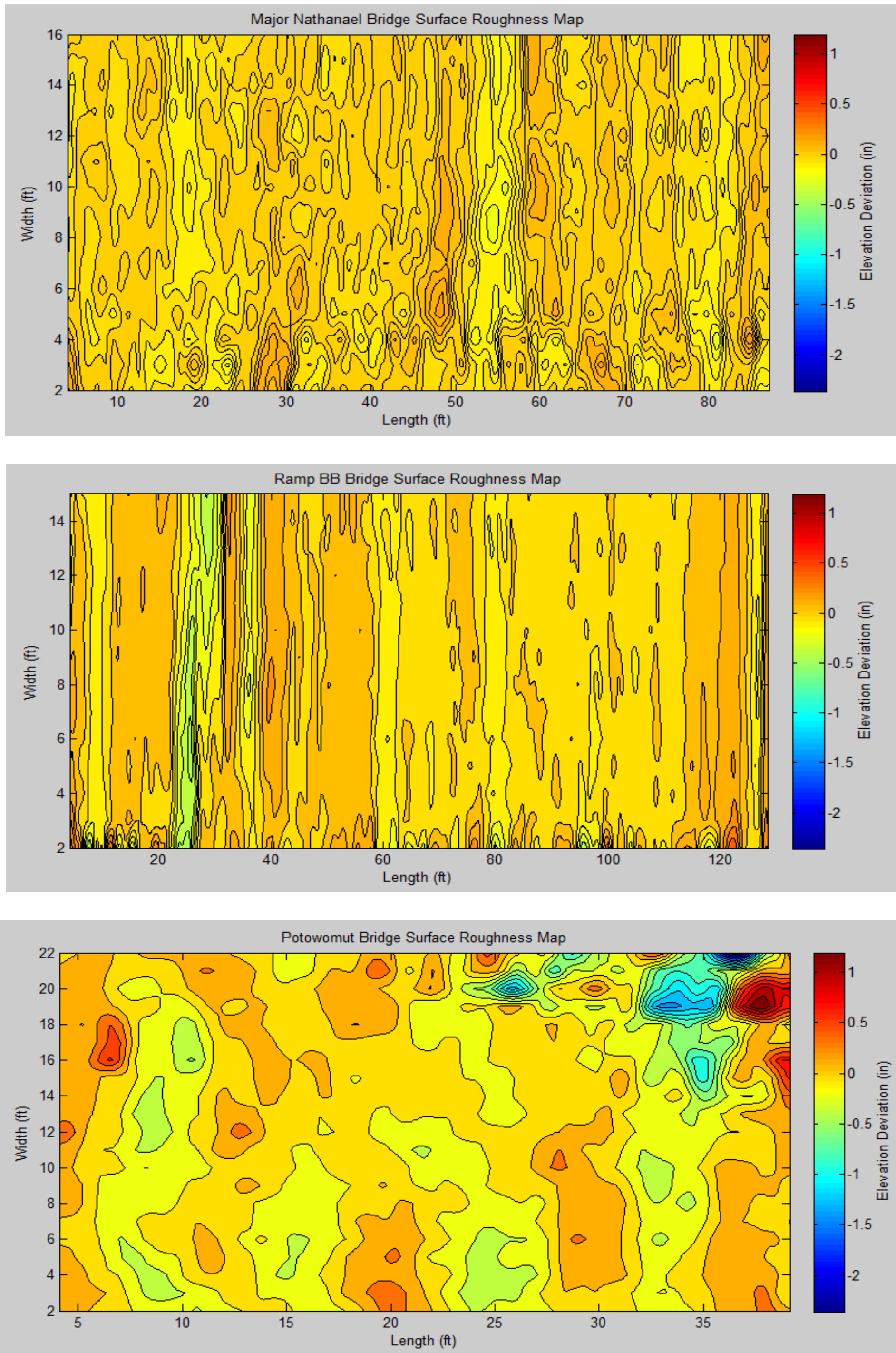


Figure 4.1.7: Surface Roughness Mapping Comparison for Major Nathanael Greene (Top), Ramp BB (Middle), and Potowomut (Bottom)

Using surface roughness data, surface mapping is useful because it shows a visual representation of deck deficiencies with more exact locations. In addition, elevation deviation and International Roughness Index (IRI) determinations allow for the quantification of deck surface deficiency and also for the comparison of one bridge deck's surface deficiencies to that of another. For example, all three bridges have a deck condition rating of 7-Good Condition, the elevation deviations of deficiencies and IRI values varied significantly. The Major Nathanael Greene Bridge in the seemingly best condition visually has an IRI value of 142.80 in/mi, indicative of new pavement, with isolated surface deficiencies ranging only from -0.2 to 0.17 inches in elevation deviation. Contrastingly the Potowomut Bridge in the seemingly worst condition visually has an IRI value of 539.97 in/mi, indicative of damaged pavement, with surface deficiencies ranging from -2.36 to 1.19 inches in elevation deviation.

4.2 Ground Penetrating Radar

Once GPR data were collected, they were exported and processed using RADAN [31] software developed by GSSI. RADAN allows users to view, manipulate, and locate buried objects such as steel rebar. Within the software, individual longitudinal profiles are appended together as a 3D batch of files, where information such as testing direction, bridge length, distance from curb, distance between profiles, skew, and start and end locations can be specified. The files were first corrected to set the position of zero time at the surface of the deck. For various reasons including the altered shape of the emitted wave and the reduced frequency of the signal in the air between the antenna and the ground surface, the arrival time of a reflected wave off of a target will also shift to a later time. Thus, the first reflection is not at the ground surface [53]. In this analysis, the first positive peak of the signal is used for time zero correction as shown in Figure 4.2.1.

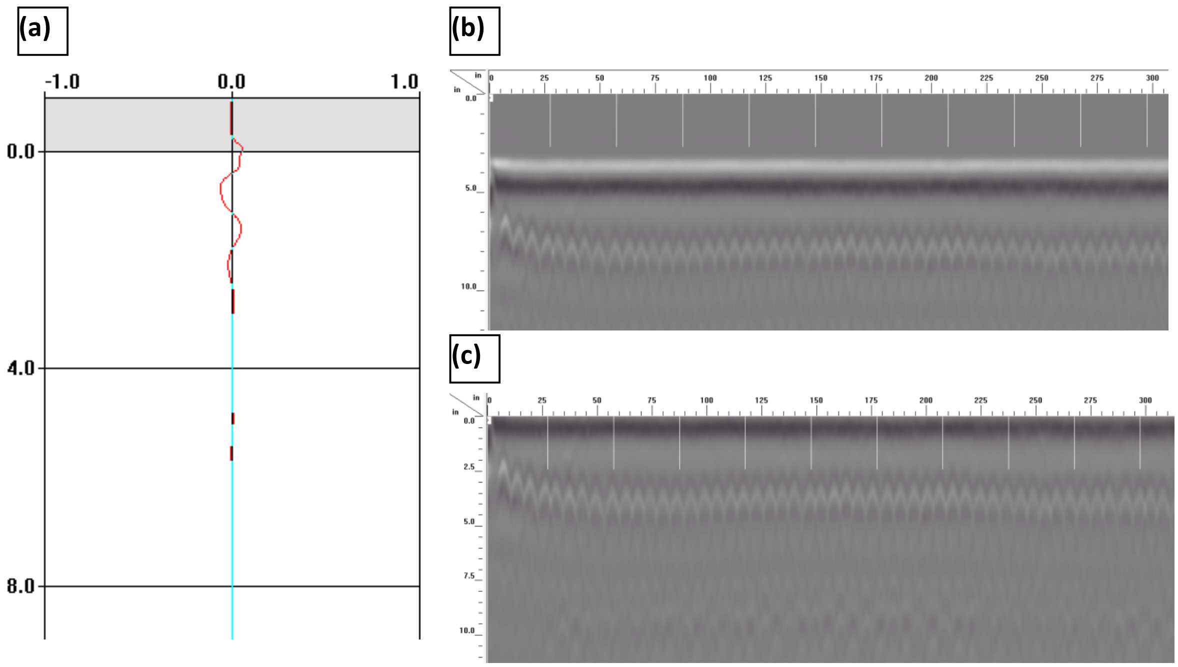


Figure 4.2.1: Time Zero Correction (a) A-scan (b) Data Before Time Zero Correction (c) Data After Time Zero Correction for the Major Nathanael Greene Bridge

Following time zero correction, the data were migrated to better differentiate reinforcement bar location. The migration signal processing technique is used to collapse the hyperbolic features to a more singular point representative of the rebar, depending on an optimal choice of signal velocity [3]. The software contains an auto-target function that automatically scans each image and locates the peak of each hyperbola and marks it as the location of rebar. Depending on the clarity of the data, however, more often than not this option misidentified the location of rebar. As a result, rebar was located manually for nearly all scans. This consumed considerable amount of time but provided the most reliable data.

RIDOT allowed cores to be taken from the deck of the Potowomut Bridge. These are shown in Figure 4.2.2 and Figure 4.2.3. The depth of the rebar was measured and used as ground truth points to validate the location of the rebar determined from GPR. Results are reported in Table 4.1.1. A difference of only 1% was found. Thus, information from GPR can be used reliably.

Figure 4.2.2: Potowomut Bridge Core 1 (X = 7 ft, Y = 25 ft)

Figure 4.2.3: Potowomut Bridge Core 2 (X = 4 ft, Y = 15 ft)

Table 4.2.1: Concrete Core Details from Potowomut Bridge

Core	Visual Observations	Normalized Amplitude Difference from Least Deteriorated Point (dB)	Depth to Rebar (in)	Depth from RADAN (in)	% Difference
1	Concrete and rebar are free from delamination	-5.81	3.50	3.52	0.57%
2	Severe delamination of concrete above rebar, minor corrosion of rebar	-14.68	2.50	2.53	1.2%

The strength (or normalized amplitude) of the rebar found in Core 2, which exhibited severe delamination of the concrete cover above rebar and minor corrosion of rebar, varied -14.68 dB from the least deteriorated point of the bridge deck, compared to that of Core 1, free of delamination, which varied -5.81 dB. This demonstrates with larger amplitude attenuation (from the least deteriorated point, or the maximum amplitude), there is a higher likelihood of deterioration [46].

Once the rebar was located for each bridge deck (i.e. obtain X-, Y-, and Z- coordinates), the variations in concrete cover (Z-coordinate) and rebar spacing (X-coordinate) were assessed and compared to as-built drawings. Figures 4.2.4-4.2.6 present the distribution of the concrete cover and rebar spacing for each bridge, respectively. Data are also summarized in Table 4.2.2.

Table 4.2.2: Comparison of Concrete Cover and Rebar Spacing obtained from GPR with As-Built Values

Bridge	Concrete Cover from Plans (in)	Average Concrete Cover from GPR (in)	% Difference	Rebar Spacing from Plans (in)	Average Rebar Spacing from GPR (in)	% Difference
Major Nathanael Greene Bridge	3	3.27	9.00	6	6.29	4.83
Ramp BB Bridge	2.5	2.79	11.60	8	7.34	8.25
Potowomut Bridge	2	3.15	57.50	N/A	6.42	N/A

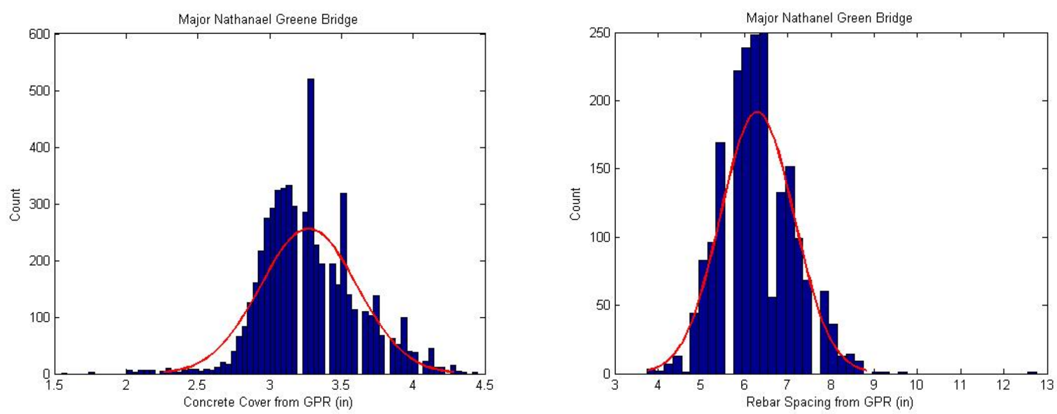


Figure 4.2.4: Concrete Cover and Rebar Spacing Distribution for Major Nathanael Greene

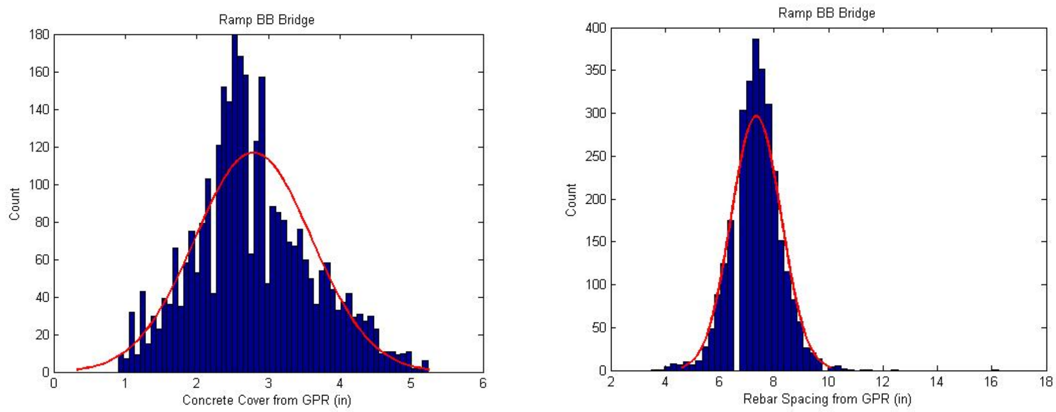


Figure 4.2.5: Concrete Cover and Rebar Spacing Distribution for Ramp BB

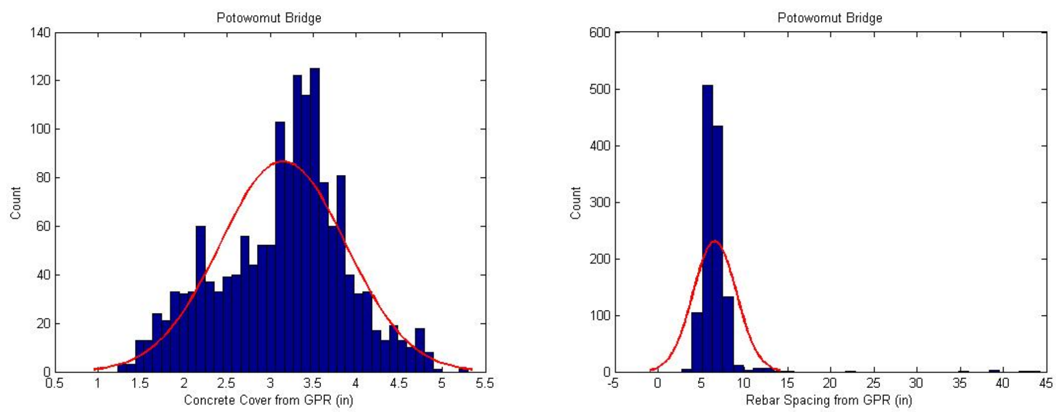


Figure 4.2.6: Concrete Cover and Rebar Spacing Distribution for Potowomut

The average concrete cover for Major Nathanael Greene and Ramp BB is determined to have a small percent difference of 9.00 and 11.60 respectively when compared to information provided in as-built plans. For Potowomut, the percent difference is found to be 57.50 %, likely due to areas of exposed rebar and thick asphalt patches that significantly varied in concrete cover. The average rebar spacing for Major Nathanael Greene and Ramp BB is determined to have a small percent difference of 4.83 and 8.25 percent respectively when compared to information provided in as-built plans. For Potowomut, rebar spacing was not found in the as-built plans, however isolated areas with large rebar spacing can be attributed to areas of missing rebar. Upon comparing the average concrete cover and rebar spacing values obtained from GPR testing and RADAN to those found in as-built plans for each bridge, it is determined that the data from GPR testing can be used reliably.

4.3 Deterioration Threshold in GPR Data

Examples of rebar locations within GPR scans are presented in Figure 4.3.1 for scans with well-defined hyperbolas as in the case of Major Nathanael Greene and for scans with poorly-defined hyperbolas as in Potowomut Bridge. Poorly defined hyperbolas exhibit lower reflection amplitude, or higher amplitude attenuation, and are often an indication of deterioration [9, 11].

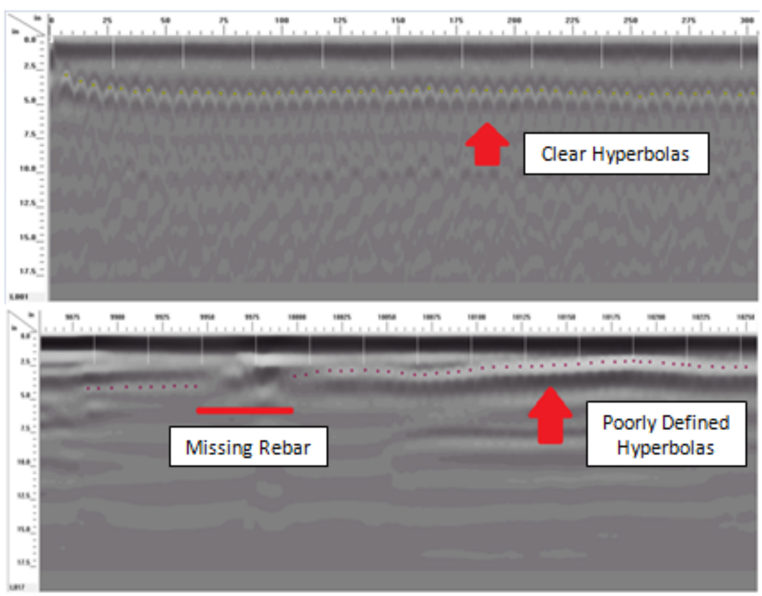
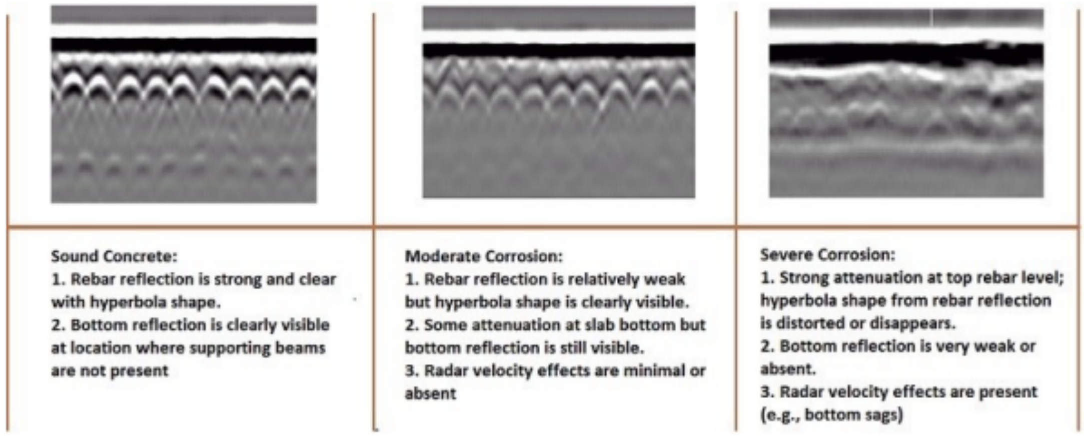


Figure 4.3.1: Rebar Location in GPR Scans from (a) Major Nathanael Greene Bridge and (b) Potowomut Bridge



Source: Dinh, 2014

Figure 4.3.2: Rebar Clarity as Deterioration Indication

Using image-based analysis, the clarity of the rebar hyperbolas in a scan can be used to mark attenuated areas by an experienced analyst. When visually inspecting each scan, areas of noticeable deterioration can be categorized by severity. The percentages of little, moderate, and severe deterioration can then be calculated for the entire bridge deck or test section, and deterioration can be mapped as demonstrated in reports by Dinh & Zayed (2016) [11] and Tarussov et al. (2013) [50].

In all case studies performed by Tarrusov, where GPR data were analyzed using an image-based analysis approach and correlated with extracted cores and chain-drag delamination surveys, the visual analysis of the GPR profiles proved to be reasonably precise in mapping in-situ condition of the concrete structure. Several analysis techniques are visual or auditory: visual concrete inspection, hammer testing, chain-drag, etc. These “subjective” methods are accepted and often provide more information than other kinds of numerical tests. There is no reason to discard an accurate technique simply because it does not quantify the output [50].

When GPR data are collected, most often they are translated into a graphical deterioration map, as demonstrated in reports by Parrillo et al. (2005) [28], and Wang et al. (2011) [51]. With these deterioration maps, the areas of deterioration are identified where amplitude, or strength of returned signal, values vary most from the "least deteriorated point", or that with the strongest return signal. In doing so, the described deterioration areas are only deteriorated in relation to the best part of the bridge deck. Though a deterioration threshold is most often defined subjectively, by the operator, considering bridge deck age, visual deck condition and the signal change intensity [21], in a report by Zou (2013), the deterioration threshold is typically defined as a single amplitude attenuation magnitude within the data range (i.e. -8 dB for GPR), and for measurements beyond this threshold there is a high probability of deterioration [55].

In the American Society for Testing Materials (ASTM) Standard D6087-08 "Standard Test Method for Evaluating Asphalt-Covered Concrete Bridge Decks Using Ground Penetrating Radar", Section 7.3.4.2 defines scans containing reflection amplitude less than 6 to 8 dB below the maximum reflection amplitudes recorded typically correspond to deterioration detected using other information such as bridge deck bottom inspection results, core data when possible, and results from other deterioration assessment techniques to refine the threshold value [46]. Limitations with using numerical amplitude analysis to quantify deterioration arise if the "least deteriorated point" of the bridge deck is in fact deteriorated itself. Mapped and quantified deterioration then only describes the amount of deterioration in relation to the least deteriorated point, a point of unknown deterioration.

In order to refine the threshold value, corroborative testing methods can be performed. Many have studied the correlation of ground penetrating radar findings with those from other NDE technologies like electrical resistivity, half-cell potential, and impact-echo, as well as simplistic traditional methods like hammer sounding and chain dragging. For example, ground penetrating radar testing can be performed to determine concrete degradation, and half-cell potential testing can be performed to determine active corrosion rating. GPR findings and HCP measurements have been found to correlate well, and can be used in conjunction to develop a deterioration threshold [35, 19, 20, 22, 23]. With the development of the deterioration threshold unique to each bridge deck, through corroborative NDE testing, the percent deterioration can be calculated using the following equation provided in the ASTM Standard [46]:

$$X_{tn} = [(W_{dt}) / (W_{dt} + W_{st})]\ [100]$$

Where:

X_{tn} = percent deteriorated in a GPR inspection pass, n, at or above top steel

n = GPR inspection pass identification number

W_{dt} = concrete deteriorated at or above top steel, obtained from reflection amplitude below deterioration threshold value

W_{st} = sound concrete at top steel, obtained from reflection amplitudes above the deterioration threshold value

When evaluating the correlation between ground penetrating radar and chain drag, Yehia et al. (2007) [52] tested two concrete bridge decks and found that GPR testing indicated different deterioration findings than did chain drag. For one bridge deck GPR found 35% to be deteriorated compared to 21% found using chain drag, and for the second bridge 21% compared to 13% respectively. Because chain dragging and hammer sounding are techniques capable of detecting subsurface deterioration only in its moderate to severe stages, it is evident that a fuller picture of subsurface deterioration, including earlier stage deterioration, is obtained through the use of advanced technologies like ground penetrating radar. Limitations with establishing a deterioration threshold using correlation with other NDE testing methods arise as this approach can be incredibly time consuming and labor intensive, and may require extensive expertise for the collection, processing, and interpretation of data.

Table 4.3.1: Limitations of Current Deterioration Assessment Methods

Methods for Assessing Deterioration	**Limitations**
Image-based (Visual Clarity of Scan)	• Subjective interpretation • Estimates location and severity of deterioration
Numerical Amplitude of GPR Data (ASTM)	• Highly variable • Attenuation range, no exact threshold universal for all bridges • Highly dependent on proper data collection
GPR + Additional NDE Method (Deterioration Threshold)	• Needs corroboration from other NDE testing methods, therefore time consuming and labor intensive • Threshold is unique to each bridge deck, rather than universal
Comparison Analysis Over Time (Mean Attenuation)	• More than one inspection required for comparison • For best results, the dimensions of the test section and data collection procedure must remain the same

In order to assess deterioration in concrete bridge decks, a comparison analysis over time approach was chosen. Though the collection of more data is required to compare condition over time, this type of analysis allows for the evaluation of change in subsurface deterioration over time without the need for extensive data processing, corroborative testing methods, or deterioration thresholds. Data were collected for three bridges of varying visual deck condition over a two-year time period using ground penetrating radar. For each bridge, GPR normalized amplitude attenuation was plotted in a contour map, displaying the difference in amplitude from the deck's least deteriorated point for both the original and secondary test. A contour map of the change in attenuation over the two-year time period was also generated, to display change in subsurface deterioration over time.

Without GPR data obtained when a bridge was first constructed, and without costly and time consuming corroborative test methods, incorporating GPR testing into routine bridge inspection still allows management agencies to better assess subsurface deterioration. Not only can potentially hazardous deterioration hidden beneath the surface be viewed, but also the rate of subsurface deterioration through comparison analysis over time can be analyzed to determine what smaller and more cost-effective improvement strategies should be implemented, and when to preserve the bridge deck and extend its lifespan.

4.4 Ground Penetrating Radar Subsurface Deterioration Mapping

Normalized amplitude attenuation data obtained from GPR field testing and RADAN processing were imported into Surfer [48], a 2D and 3D mapping, modeling, and analysis software program, to generate GPR subsurface deterioration maps for both initial and secondary testing. It is important to note the normalized amplitude difference (attenuation) scale is unique to each bridge and to each test. In first keeping the scale unique to each bridge and to each test, the areas of most attenuation will always be red in color, and the areas of least attenuation will be purple or blue in color. Without comparing one map to another, this helps to visually display the subsurface condition of a bridge deck at the present time, and identify the range of normalized amplitude difference, where a larger range is indicative of more deterioration. The 2015 and 2017 subsurface deterioration maps for Major Nathanael Greene, Ramp BB, and Potowomut are pictured below. The change in range of normalized amplitude is included in Table 4.4.1.

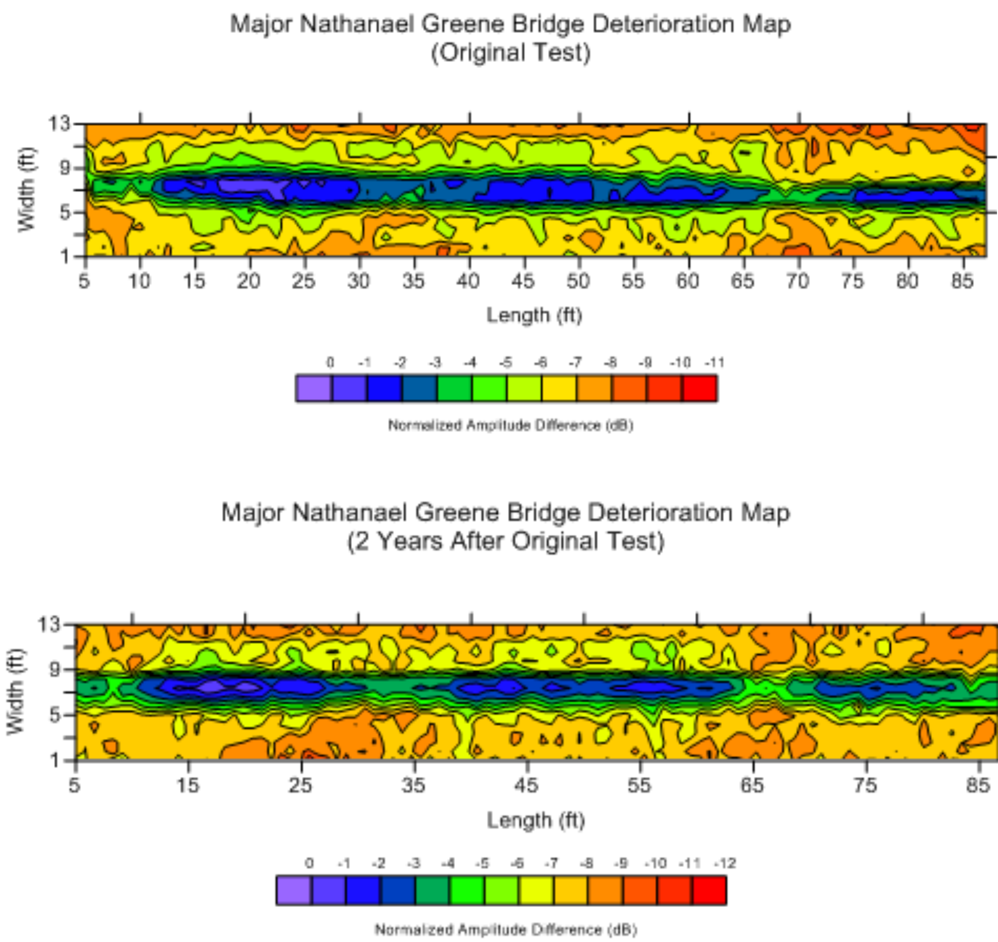
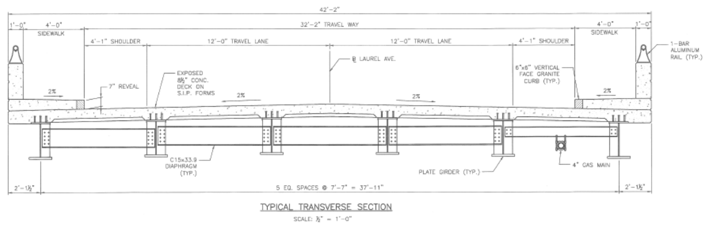


Figure 4.4.1: Major Nathanael Greene Bridge GPR Subsurface Deterioration Map

A typical transverse section of the Major Nathanael Greene Bridge is pictured below, obtained from as-built plans. The test section included GPR collection over one girder, displayed longitudinally along the middle of the deterioration maps.



Source: Rhode Island Department of Transportation

Figure 4.4.2: Major Nathanael Greene Typical Transverse Section

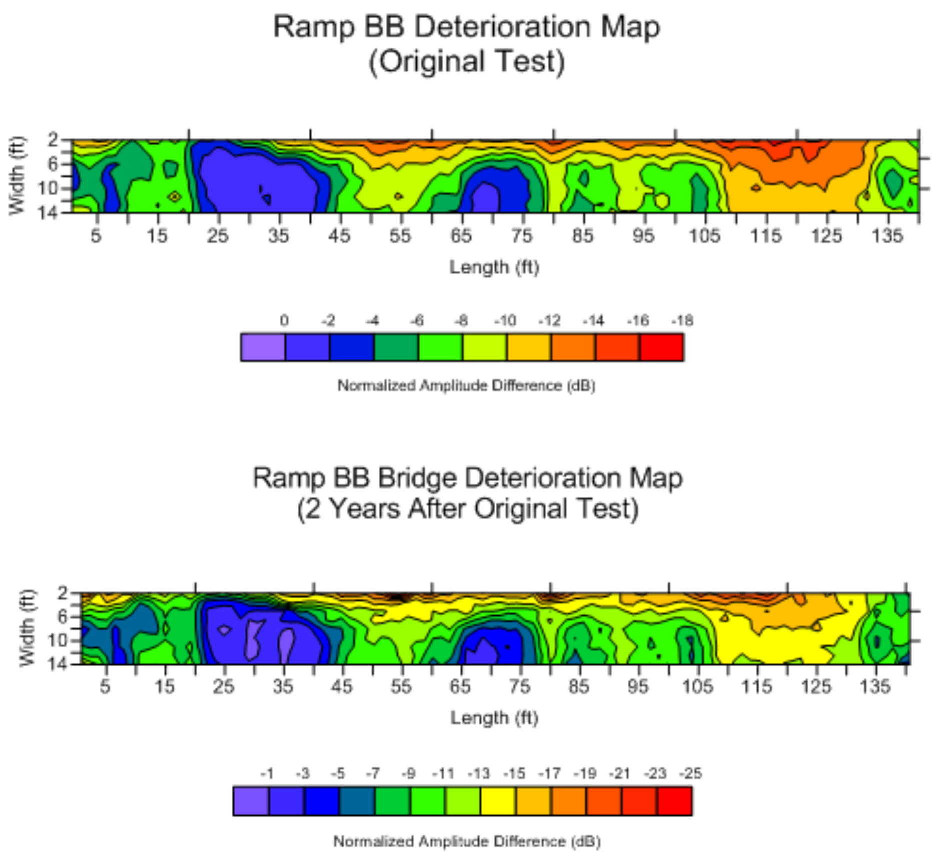
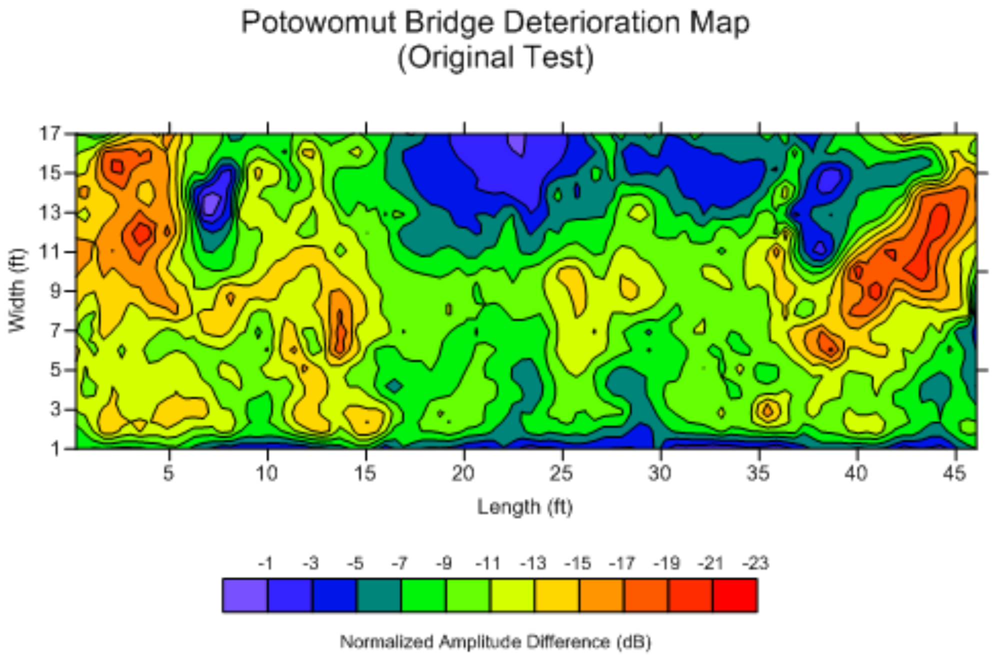


Figure 4.4.3: Ramp BB Bridge GPR Subsurface Deterioration Map



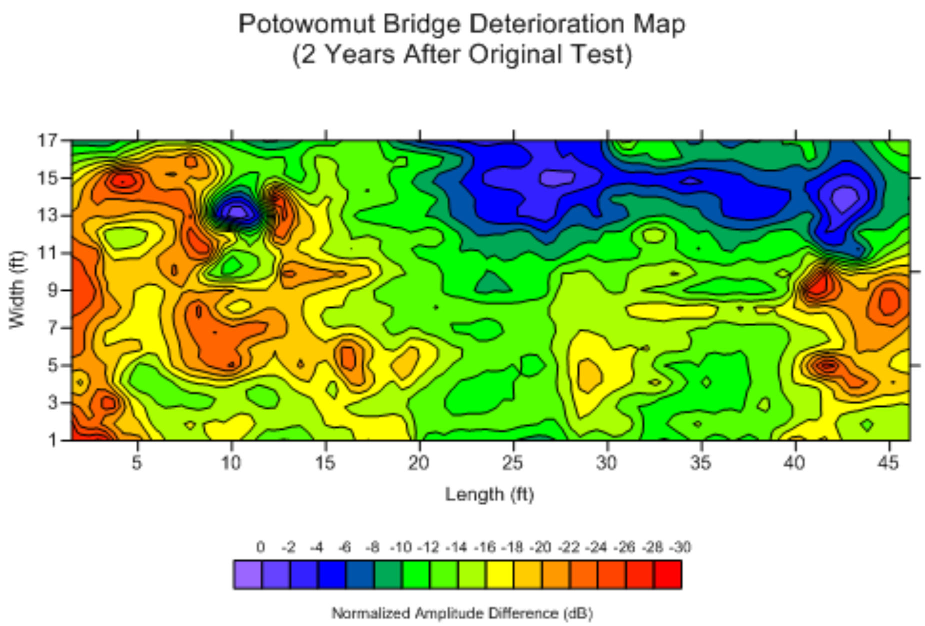


Figure 4.4.5: Potowomut Bridge GPR Subsurface Deterioration Map

Table 4.4.1: Normalized Amplitude Range

Bridge	2015 Normalized Amplitude Range (dB)	2017 Normalized Amplitude Range (dB)	Change in Normalized Amplitude Range over 2 Years (dB)
Major Nathanael Greene	11	12	1
Ramp BB	18	25	7
Potowomut	23	30	7

Over the two-year time period, Major Nathanael Greene has experienced a change in normalized amplitude range or attenuation of 1 dB, and Ramp BB and Potowomut have both experienced a change in normalized amplitude range or attenuation of 7 dB.

Though GPR subsurface mapping using normalized amplitude difference scales is helpful in determining areas of attenuation unique to that bridge deck at that specific time, creating a universal scale per bridge deck allows for the visual comparison of deterioration at different times. Each bridge's dataset for 2017 was re-plotted using the difference in attenuation of each point from the least deteriorated point of the 2015 dataset. This allowed for the display of change in attenuation from 2015 to 2017. Subsurface attenuation maps with a corrected scale for each bridge deck, for Major Nathanael Greene, Ramp BB, and Potowomut are pictured below.

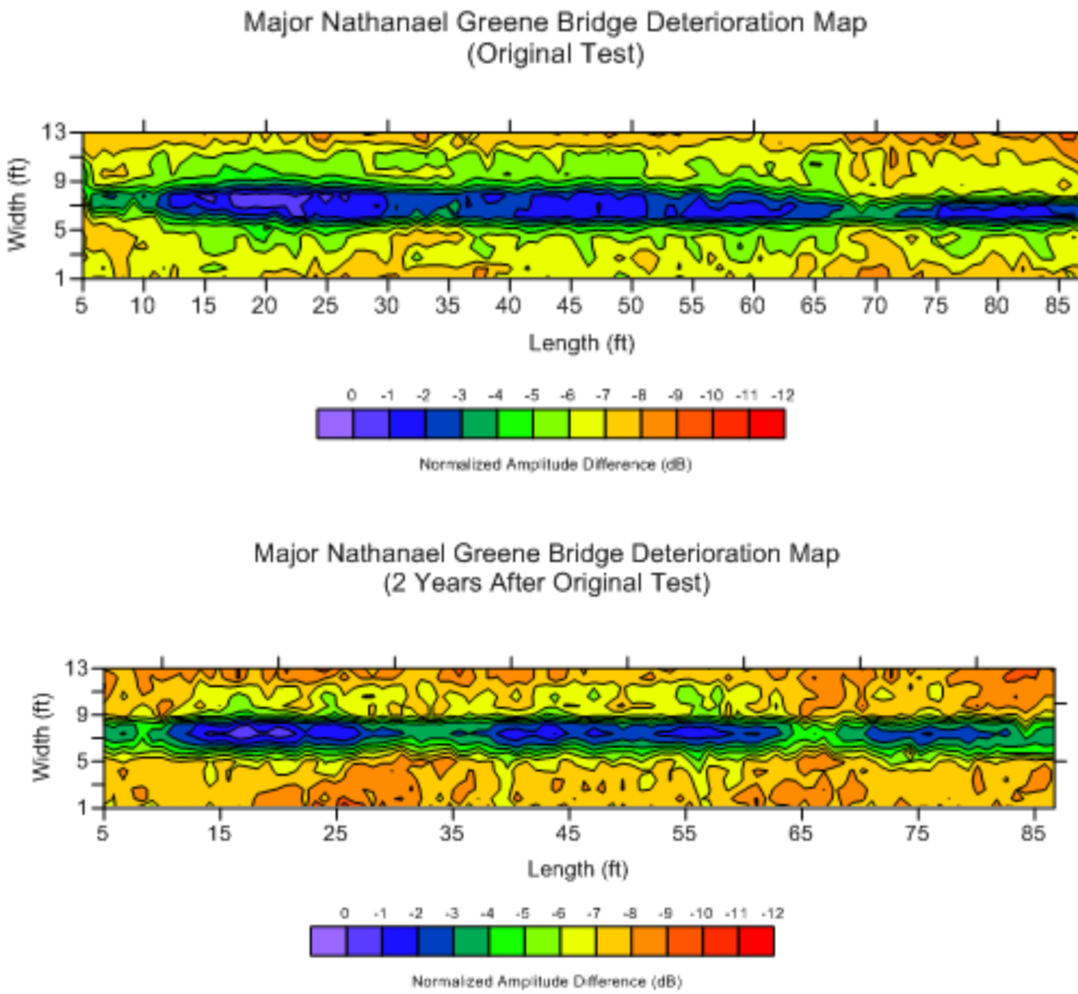
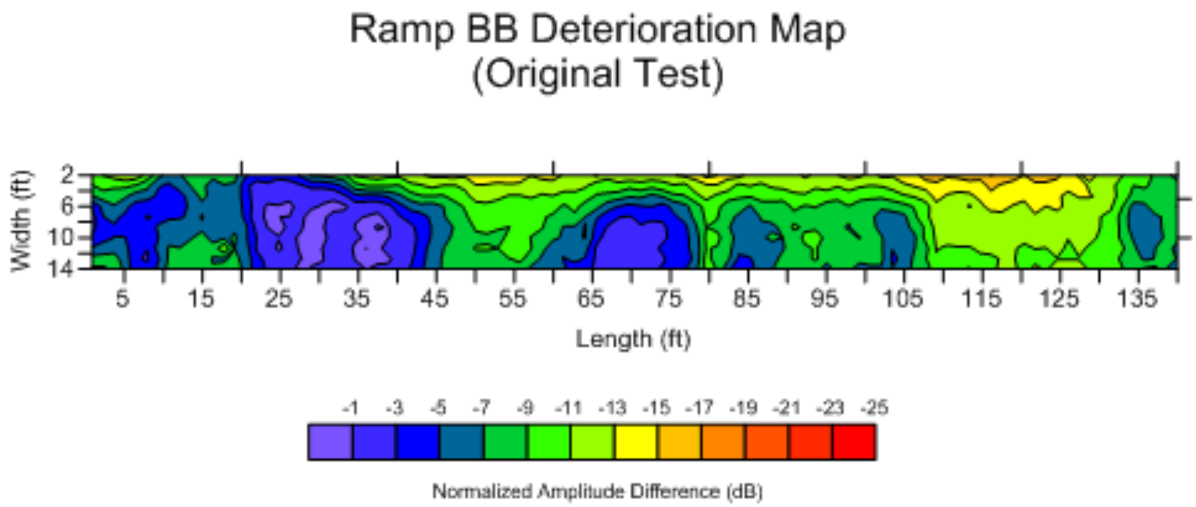


Figure 4.4.6: Major Nathanael Greene Bridge GPR Subsurface Deterioration Map Comparison

Displayed in Figure 4.4.6, the Major Nathanael Greene Bridge has changed slightly in subsurface condition over the two-year time period. Areas green and yellow in color surrounding the girder have experienced a decrease in normalized amplitude, or an increase in amplitude attenuation, and are now yellow and orange in color.



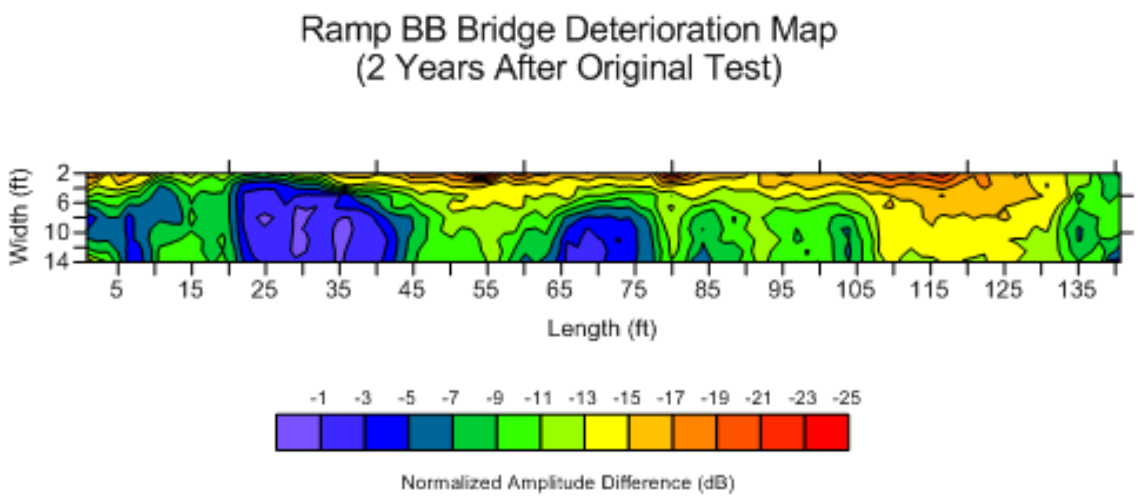


Figure 4.4.7: Ramp BB Bridge GPR Subsurface Deterioration Map Comparison

Displayed in Figure 4.4.7, the Ramp BB Bridge has changed moderately in subsurface condition over the two-year time period. Areas green and yellow in color nearing the curb have experienced a decrease in normalized amplitude, or an increase in amplitude attenuation, and are now yellow and orange in color.

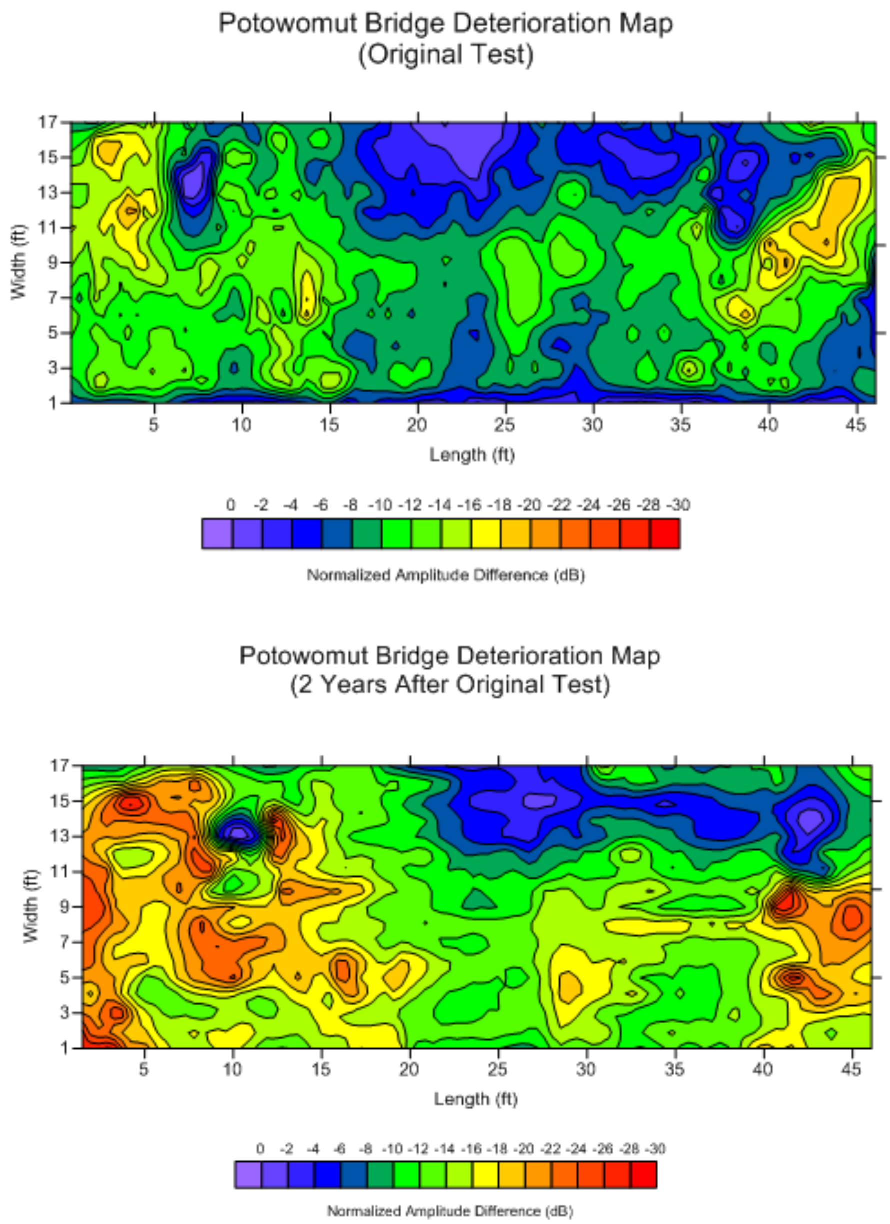


Figure 4.4.8: Potowomut Bridge GPR Subsurface Deterioration Map Comparison

Displayed in Figure 4.4.8, the Potowomut Bridge has changed significantly in subsurface condition over the two-year time period. Areas green in color at the beginning and end of the test section, near the joints, have experienced a decrease in normalized amplitude, or an increase in amplitude attenuation, and are now yellow and orange in color.

While normalized amplitude range indicates the range between the most deteriorated point and the least deteriorated point, the mean of the normalized amplitude attenuation better indicates change in overall subsurface deterioration over time, and the standard deviation better indicates the distribution of points within the range. For example, though both Ramp BB and Potowomut increased 7 dB in normalized amplitude range over the two-year time period, it is evident upon comparison of the subsurface maps that overall, the Potowomut Bridge has deteriorated more than the Ramp BB Bridge. A great change in normalized amplitude range, or a wider spectrum of values, can be attributed either to widespread deterioration across the entirety of a bridge deck, or even just one single, small area of isolated deterioration. It is therefore important to determine the mean and standard deviation of the normalized amplitude attenuation, to better evaluate subsurface deterioration.

When keeping the attenuation scale constant between the initial and secondary testing, it is visually apparent what areas of the bridge deck are deteriorating, and to what extent. Further analyzing the GPR data, using the ASTM Standard and statistical parameters, the change in deterioration reported as an overall percentage and as a percent change of initial condition, was determined.

Table 4.4.2: Normalized Amplitude Attenuation Parameters

Bridge	Mean Attenuation (dB)	Change in Mean Attenuation Over Time (%)	Standard Deviation (dB)	Change in Standard Deviation Over Time (%)
Major Nathanael Greene (2015)	-5.70	14.04	2.03	2.46
Major Nathanael Greene (2017)	-6.50		2.08	
Ramp BB (2015)	-7.72	31.09	3.80	22.37
Ramp BB (2017)	-10.12		4.65	
Potowomut (2015)	-9.67	43.85	3.90	30.52
Potowomut (2017)	-13.91		5.09	

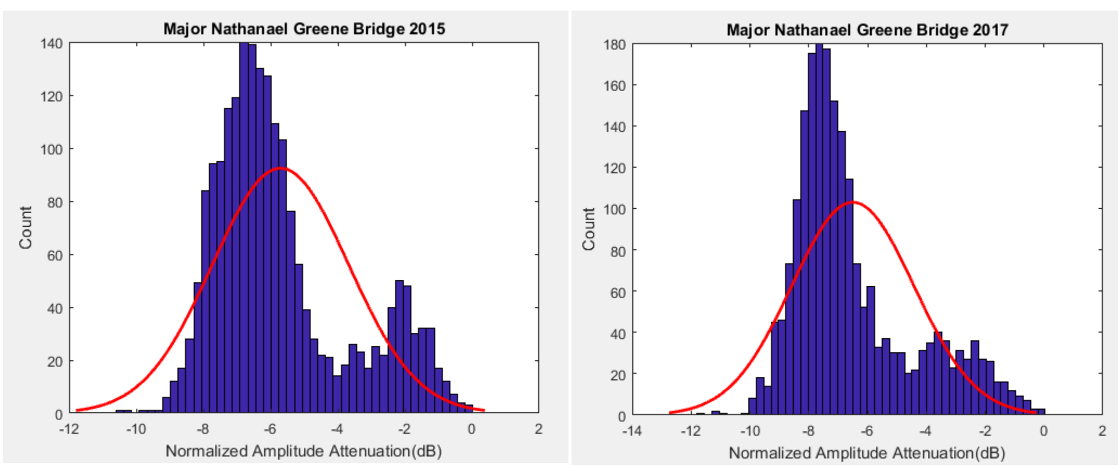


Figure 4.4.9: Major Nathanael Greene Attenuation Histograms for 2015 and 2017

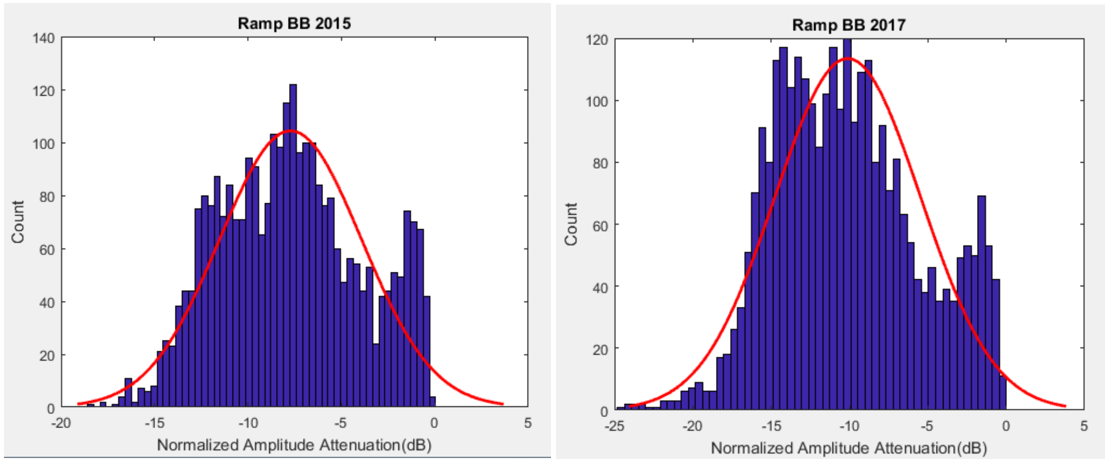


Figure 4.4.10: Ramp BB Attenuation Histograms for 2015 and 2017

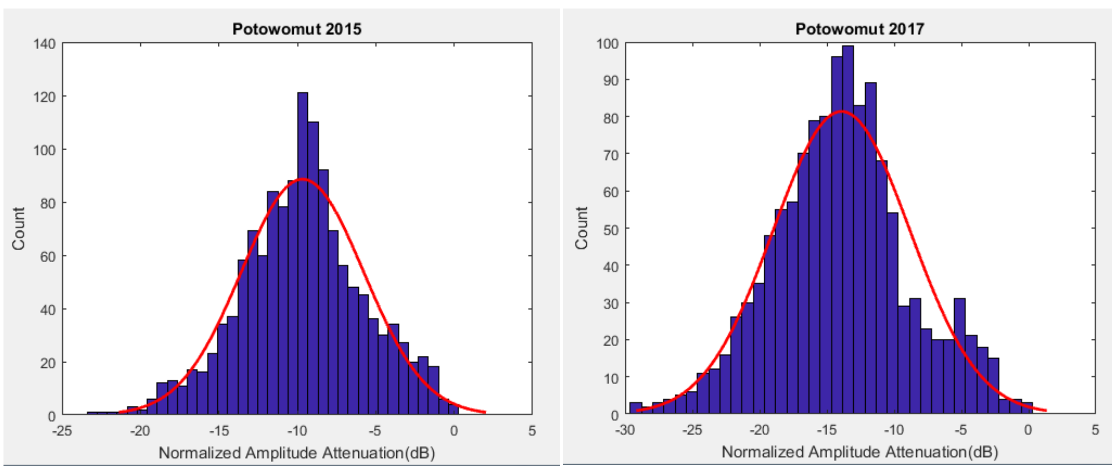


Figure 4.4.11: Potowomut Attenuation Histograms for 2015 and 2017

Table 4.4.3: Percent Deteriorated Using ASTM Standard

Bridge	% Deteriorated using -8dB Threshold (%)	Change in % Deterioration Over 2 Years (%)
Major Nathanael Greene (2015)	6.74	15.73
Major Nathanael Greene (2017)	22.47	
Ramp BB (2015)	48.40	21.17
Ramp BB (2017)	69.57	
Potowomut (2015)	69.70	17.82
Potowomut (2017)	87.52	

Initial GPR testing in 2015 was performed concurrently by Dr. Nicole Martino of Roger Williams University, along with impact-echo (IE) testing to determine percent deterioration. She had found that Major Nathanael Greene was 0% deteriorated and Potowomut was 70% deteriorated. This information can be compared to the 2015 percent deteriorated value calculated using the -8 dB threshold from the ASTM Standard, to determine a 12.33 and 0.43 percent error for the Major Nathanael Greene and Potowomut Bridge, respectively.

Using the ASTM standard deterioration threshold, it is evident that each bridge has experienced increased subsurface deterioration over the two-year time period. When analyzing the change in mean attenuation over the same two-year time period, Major Nathanael Greene, Ramp BB, and Potowomut have increased by 14.04, 31.09, and 43.85 percent respectively. Without using a deterioration threshold, change in mean attenuation describes the percentage by which overall subsurface condition has gotten worse over time. Major Nathanael Greene has worsened in subsurface condition by 14.04% over two years, Ramp BB by 31.09% over two years, and Potowomut by 43.85% over two years, from 2015 to 2017. This information is meaningful in providing rate of subsurface deterioration, to determine the best time to make improvement efforts for preservation purposes, without the need for a deterioration threshold.

CHAPTER 5

ANALYSIS OF BRIDGE CONDITION RATINGS

As evident from the variance in visible surface conditions of the three bridges, though given an equal bridge deck condition rating during inspection in 2015 as reported by NBI, bridge deck condition can be more accurately assessed and reported. The deck condition ratings for all concrete bridge decks in Rhode Island were analyzed to determine not only the condition of Rhode Island concrete bridge decks, but also the rate of deterioration based on initial condition. Determining the rate of deterioration based on condition can aid in determining what management strategies should be implemented and when, to extend the service life of the infrastructure and to make driving safer for the public.

5.1 Rhode Island Concrete Bridge Deck Condition Rating

Using data obtained from the National Bridge Inventory, it was determined in the state of Rhode Island over a ten-year period, 1,110 concrete bridge decks have reported bridge deck inspection data. Over this ten-year period, from 2007 to 2016, bridges that did not have yearly data for each of the 10 years were removed from the dataset. Of the 1,110 concrete bridge decks, 494 bridges have deck condition data continuously for each of the 10 years, from 2007 to 2016. Over the ten-year period, any bridges that were reconstructed were removed from the dataset. Of the 494 bridges that have 10 years worth of deck condition data, 429 bridges were not reconstructed within the ten-year period.

From the dataset containing 429 non-reconstructed concrete bridge decks in Rhode Island with 10 years worth of continuous yearly data, bridge deck condition was analyzed first to determine initial rating.

Table 5.1.1: Rhode Island Concrete Bridge Deck Condition Rating Data

Initial NBI Deck Condition Rating	Bridge Count	% of Total
8	31	7.23%
7	159	37.06%
6	182	42.42%
5	52	12.12%
4	5	1.17%
Total	429	100%

As displayed in Table 5.1.1, the majority of Rhode Island concrete bridge decks in this dataset are rated 7-Good (37.06%) and 6-Satisfactory (42.42%), collectively making up nearly 80% of the dataset.

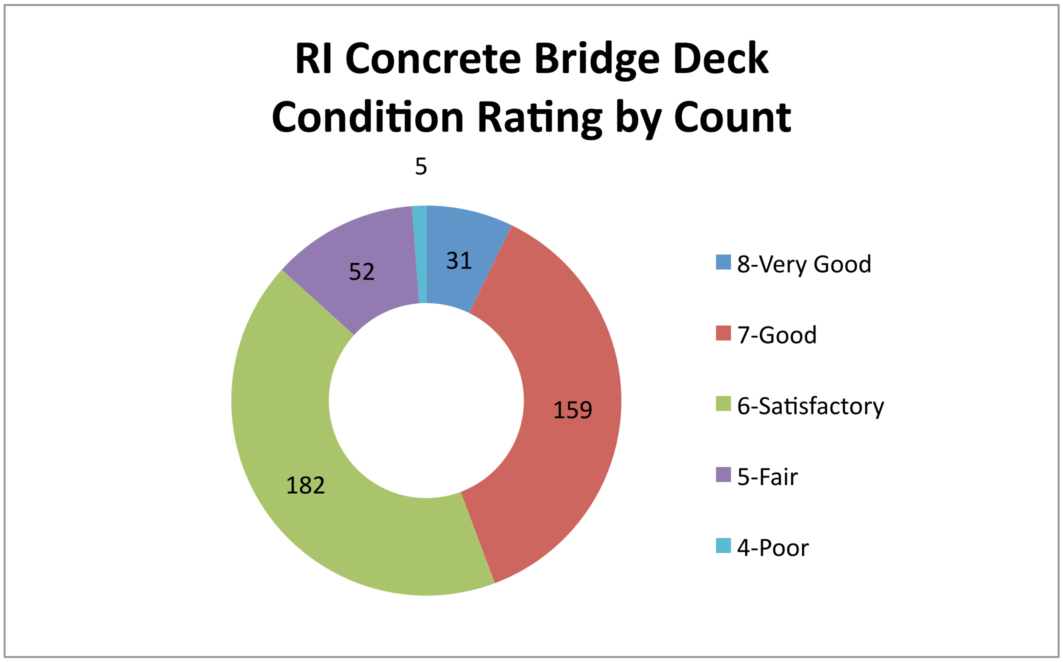


Figure 5.1.1: RI Concrete Bridge Deck Condition Rating by Count

With 42.42% of concrete bridge decks in the dataset nearing below satisfactory conditions, it is important to determine the rate of deterioration based on initial deck condition in order to decide what improvements should be made and when to preserve good deck condition. In most cases, the cost to maintain a concrete bridge deck in good condition is significantly less than the cost to repair a concrete bridge deck in fair condition. The dataset was then analyzed to determine at what rate deterioration occurs, based on initial deck condition.

Table 5.1.2: Percentage Change in Deck Condition Rating of RI Concrete Bridge Decks over 10 Years

% of Total	Initial Deck Condition Rating	% No Change in Condition Rating	% Decrease 1 Condition Rating	% Decrease 2 Condition Ratings	% Decrease 3 Condition Ratings
7.23%	8	6.45%	93.55%	9.68%	0%
37.06%	7	54.72%	43.40%	5.66%	0.63%
42.42%	6	73.08%	24.18%	4.95%	0%
12.12%	5	73.08%	25%	0%	1.92%
1.17%	4	80%	20%	0%	0%

The information provided in Table 5.1.2 describes that for example, of the 37.06% of bridges in the dataset (Rhode Island Concrete Bridge Decks that have not been reconstructed and have continuous yearly data over a ten-year period) that had an initial deck condition rating of 7, 54.72% have no change in condition rating, 43.40% decrease by 1 condition rating, 5.66% also decrease by 2 condition ratings, and 0.63% also decrease by 3 condition ratings over a 10 year period. This information describes that with a higher initial deck condition rating there is a greater percentage of decreased deck condition rating over the ten-year period. In other words, a bridge deck with an initial condition of 8 is more likely to decrease in condition rating over the ten-year period than a bridge deck with an initial condition of 7, a comparison of 93.55% to 43.40% respectively. Additionally, with a lower initial deck condition rating there is a greater percentage of unchanging deck condition rating over the ten-year period.

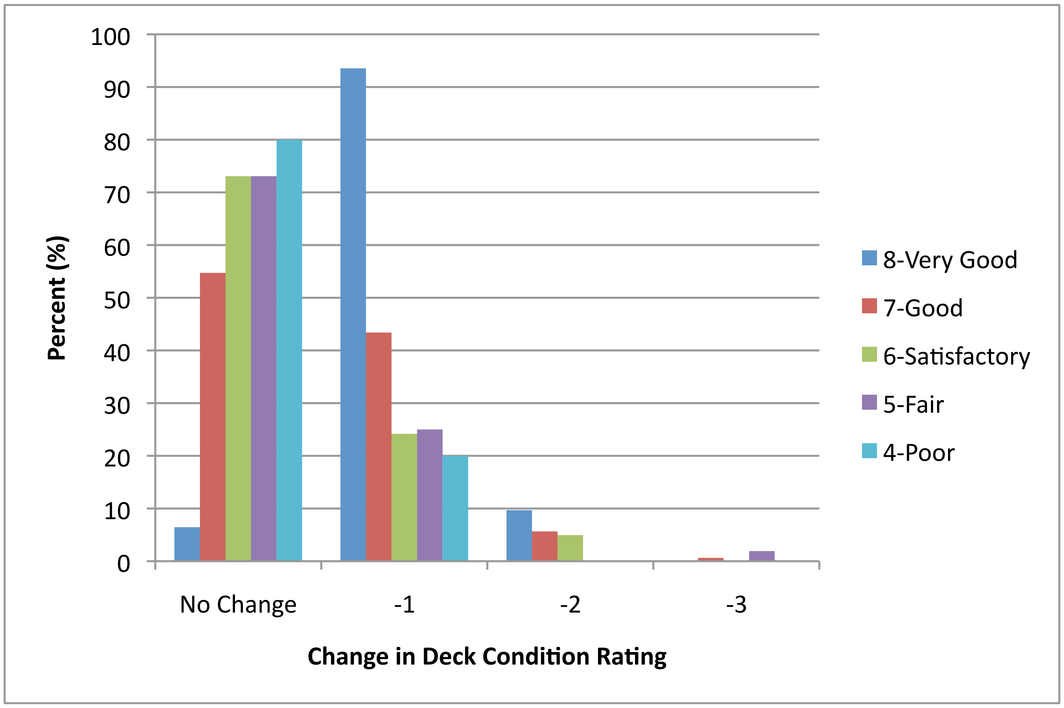


Figure 5.1.2: Change in Initial Deck Condition Rating by Percent Over 10 Years Based on Initial Condition

Bridge decks in very good condition are more likely to decrease in condition rating than bridge decks in fair condition, because the difference in condition rating is not as substantial. Decreasing from a condition rating of 8 to 7 only describes a minor increase in deterioration such as light scaling, and visible tire wear. Decreasing from a condition rating of 6 to 5 on the other hand, describes additional scaling, cracking, and an increase of 20-40% deterioration. For this reason, bridges with lower initial deck condition rating are more likely to experience no change in deck condition rating over 10 years, and bridges with higher initial deck condition rating are likely to experience more change in deck condition rating over 10 years.

This information was further analyzed to determine the average amount of years it takes a bridge deck to decrease in condition rating based on initial condition.

Table 5.1.3: Average Years to Decrease in Condition Rating Based on Initial Deck Condition Rating

Count of Total	Initial Deck Condition Rating	Average Years to Decrease 1 Condition Rating	Average Years to Decrease 2 Condition Ratings	Average Years to Decrease 3 Condition Ratings
31	8	3.93	7.67	x
159	7	5.16	7.67	5.00
182	6	6.18	7.22	x
52	5	6.85	x	4.00
5	4	8	x	x

The information provided in Table 5.1.3 describes that of the 159 bridges in the dataset with an initial deck condition rating of 7, it takes an average of 5.16 years to decrease 1 condition rating, an average of 7.67 years to decrease 2 condition ratings, and an average of 5 years to decrease 3 condition ratings. When analyzing the average years to decrease 3 condition ratings, it is important to note that only 0.63% of bridges in that category decreased 3 condition ratings over 10 years. Therefore of the 0.63%, the average amount of years to decrease the 3 condition ratings was 5 years. Using Table 5.1.3 in accordance with Table 5.1.2, it can be determined that 93.55% of bridges with an initial deck condition rating of 8 decrease one condition rating over ten years at an average of 3.93 years. At a smaller percentage and a slower rate, 21.18% of bridges with an initial deck condition rating of 6 decrease one condition rating over ten years at an average of 6.18 years. Though a smaller percentage and slower rate, decreasing from a satisfactory condition rating to fair indicates much more significant deterioration than does decreasing from very good to good condition.

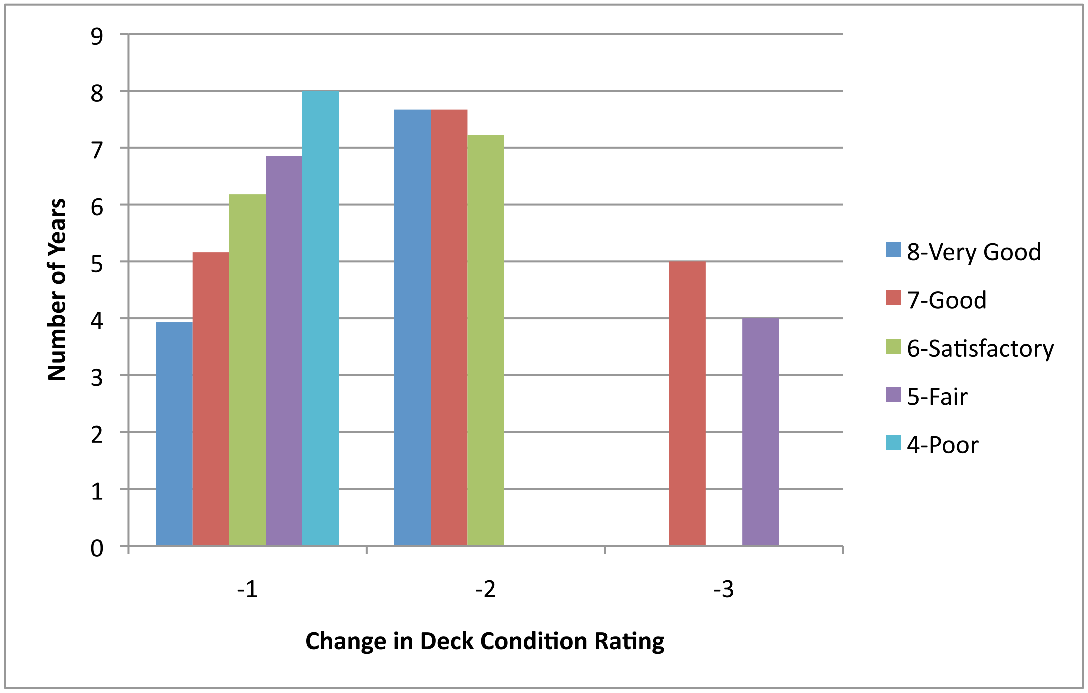


Figure 5.1.3: Average Number of Years for Change in Deck Condition Rating Based on Initial Condition

Upon comparing Figure 5.1.3 to Figure 5.1.2, it is determined that concrete bridge decks in better initial condition are more likely to worsen in reported bridge deck condition in a shorter amount of time than those in poorer initial condition. It is also determined that concrete bridge decks in poorer initial condition are less likely to experience a change in reported deck condition, and worsen in reported bridge deck condition in a longer amount of time. These findings emphasize that as concluded and reported from routine inspection, concrete bridge decks in better initial condition are more likely to decrease in bridge deck condition rating, and in a shorter amount of time, than bridge decks in poorer initial condition. Though this may be true based on visible deck surface deterioration, and due to less of a change in deterioration with higher condition ratings, reporting of concrete bridge deck condition may be misleading if deck deficiency is not better assessed and subsurface deterioration is not included.

For example prior to testing in 2015, the latest deck condition rating for the newly reconstructed Major Nathanael Greene Bridge decreased from a 9-Excellent Condition to a 7-Good Condition as reported by the NBI. In that same amount of time, the Ramp BB Bridge and the Potowomut Bridge both did not change in deck condition rating, and remained 7-Good Condition. This demonstrates that a concrete bridge deck in better initial condition can decrease in reported deck condition more quickly because the change in deterioration is not substantial. Similarly, concrete bridge decks in poorer initial condition are more likely to remain unchanged in deck condition, and decrease in reported deck condition in a longer amount of time.

5.2 NBI Inspection Report Data Findings

Table 5.1.4: NBI Deck Condition Rating over Time for the 3 Bridges

Bridge	NBI Deck Condition Rating (As of Date)	NBI Deck Condition Rating (As of Date)	NBI Deck Condition Rating (As of Date)
Major Nathanael Greene	9 (2012)	7 (2015)	7 (2017)
Ramp BB	7 (2013)	7 (2015)	7 (2017)
Potowomut	7 (2013)	7 (2015)	7 (2017)

Based solely on first visual inspection, the three bridges should not have the same deck condition rating in 2015. A bridge deck with isolated hairline cracks should not have the same deck condition rating as a bridge deck with large potholes and exposed reinforcement bar. With surface roughness mapping, deck deficiencies can be better quantified, to more precisely report and further verify that a bridge deck with an IRI value of 142.80 in/mi and -0.2 to 0.17 inches in elevation deviation should not have the same deck condition rating as a bridge deck with an IRI value of 539.97 in/mi and -2.36 to 1.19 inches in elevation deviation.

From the collected and analyzed NBI bridge inspection data, the rate of deterioration based on initial condition can be estimated. For example, of the 159, or 37.06%, of concrete bridge decks in Rhode Island with an initial deck condition rating of 7-Good Condition in the dataset, 43.40% decrease by at least 1 condition rating over 10 years, in an average of just 5.16 years. With this information, it should be emphasized that smaller improvement efforts to maintain good condition are easier and more cost-effective than larger rehabilitation and replacement efforts once substantial deterioration has occurred and bridge decks are in fair or poor condition. Sealing cracks, filling potholes, or even overlays should be implemented as management strategies rather than complete deck replacement or overhaul.

In addition, the use of advanced technologies such as surface roughness and ground penetrating radar testing should be incorporated into routine bridge inspections when possible to provide more in-depth information regarding bridge deck surface and subsurface deficiency quantity, severity, and location. Regarding surface roughness testing, with detailed maps and the International Roughness Index, bridge deck deficiencies can be quantified and compared to previous condition to determine rate of visible surface deterioration, and thus what management strategies should be implemented and when before substantial deterioration continues and the subsurface is affected, creating a much bigger problem.

It is also important to properly assess subsurface deterioration within a concrete bridge deck to best report deck condition rating. Without a full picture of deterioration, the bridge deck condition rating reported in routine bridge inspections may be misleading. Concrete bridge decks in poorer initial condition could be deteriorating much more substantially than those in better initial condition, yet this is not visible during routine deck inspection and therefore not reported. Changes in subsurface condition of 31.09% and 43.85% over a 2 year time period for Ramp BB and Potowomut respectively, demonstrate that these two bridges should have decreased in deck condition rating from 2015 to 2017. When true overall deterioration is misleadingly reported, location, severity, and rate of deterioration cannot be determined. Without these determinations, the optimal time to make bridge improvements may easily be missed.

CHAPTER 6

STUDY FINDINGS

6.1 Testing Conclusions

Three Rhode Island bridges of different age, ADT, and visual deck condition were tested in 2015 using surface roughness and ground penetrating radar equipment to demonstrate that a fuller picture of concrete bridge deck deterioration can be obtained through the use of advanced technologies. The three bridges included Major Nathanael Greene Bridge in Coventry, Ramp BB Bridge in North Kingstown, and Potowomut Bridge in Warwick, Rhode Island. These bridges were then tested a second time in 2017, two years after the initial testing, to determine the change in subsurface deterioration that would likely be missed during routine bridge inspection.

During routine bridge inspections, typically performed every two years, bridge deck deficiency is reported in terms of location, quantity, and severity. Usually only regarding the visible surface of the bridge deck, reported deck deficiency during inspection can be more accurately mapped and quantified when incorporating advanced technologies like surface roughness testing. Surface roughness testing allows for mapping of deck surface elevation deviations in order to determine the International Roughness Index (IRI). With the IRI value, the overall surface roughness of the deck can be quantified and related to pavement condition experienced when driving over the bridge deck.

In addition to visual inspection of the deck surface, simplistic testing methods such as chain dragging or hammer testing are performed to estimate subsurface deterioration. Though these basic methods of testing are cost-effective, they are subjective and only detect bridge deck delamination in its moderate to severe stages, potentially too late to make preservation efforts. Ground penetrating radar testing can be incorporated into concrete bridge deck inspection to make a proper assessment of subsurface deterioration that is often hidden to the naked eye. When detected in its early stages, delamination within a bridge deck can be arrested before substantial deterioration continues. For example, with the early detection of deficient concrete cover, patching can be performed to remove deficient cover and replace with adequate cover before deterioration of the deck surface worsens, and before the reinforcement bar begins to corrode.

With surface roughness and ground penetrating radar testing, a fuller picture of concrete deck deterioration can be created and evaluated to determine what management strategies should be implemented and when, for preservation purposes. Maintaining good bridge deck condition in the present is easier and more cost-effective than rehabilitating or replacing poor bridge deck condition in the future. Testing the bridges initially, and then again two years later, allows for the comparison of current subsurface deterioration to previously recorded condition in order to determine the percent change in subsurface condition over the two-year time period. Understanding the change in subsurface deterioration that may be missed during routine bridge deck inspection is important in order to determine if bridge deck condition ratings are accurately reported.

Table 6.1.1: Study Findings

Bridge	NBI Deck Condition Rating in 2015	NBI Deck Condition Rating in 2017	IRI (in/mi) (2015)	% Change in Subsurface Condition (2017)
Major Nathanael Greene	7	7	142.80 in/mi new pavement	14.04%
Ramp BB	7	7	279.50 in/mi older pavement	31.09%
Potowomut	7	7	539.97 in/mi significantly damaged pavement	43.85%

Upon initial testing in 2015 it was first determined that the bridge decks varied in deck condition based on visual inspection. The Major Nathanael Greene Bridge had minor scaling, the Ramp BB Bridge had transverse cracks and curb erosion, and the Potowomut Bridge had major potholes with exposed rebar. Because the Major Nathanael Greene Bridge was recently reconstructed in 2012, its last deck condition rating prior to 2015 was 9-Excellent. In 2015, though the deck surface deficiency of the Major Nathanael Greene Bridge, Ramp BB Bridge, and Potowomut Bridge varied significantly, all three bridge decks had a condition rating of 7-Good as reported by NBI.

After testing each of the bridges using surface roughness equipment, it was determined that the IRI for the Major Nathanael Greene Bridge was 142.80 in/mi, representative of new pavement. The IRI for the Ramp BB Bridge was determined to be 279.50 in/mi, representative of older pavement, and the IRI for the Potowomut Bridge was determined to be 539.97 in/mi, representative of significantly damaged pavement. In quantifying the surface roughness of each candidate bridge deck, it is further verified that all three bridges should not have the same deck condition rating in 2015, as the Potowomut Bridge had an IRI value nearly double that of the Ramp BB Bridge, and four times that of Major Nathanael Greene Bridge.

Rather than using a deterioration threshold that is not yet definitive, the change in mean attenuation was analyzed to describe the percentage by which subsurface condition has gotten worse over time for each bridge. Over the same two-year time period the mean attenuation for Major Nathanael Greene, Ramp BB, and Potowomut increased by 14.04, 31.09, and 43.85 percent, respectively. This describes that Major Nathanael Greene has worsened in subsurface condition by 7.02% per year, Ramp BB by 15.55% per year, and Potowomut by 21.93% per year, from 2015 to 2017 if the rate of deterioration is assumed to be linear over time. This information is meaningful in providing rate of subsurface deterioration, to determine the best time to make improvement efforts for preservation purposes, without the need for a deterioration threshold.

After studying the data obtained from the National Bridge Inventory, it was determined that for Rhode Island concrete bridge decks, bridges in better initial condition: 1. Are more susceptible to decreasing in bridge deck condition rating, and 2. Decrease in bridge deck condition rating at a faster rate than bridges in poorer deck condition. This is demonstrated as only the Major Nathanael Greene Bridge decreased in condition rating from 2012 to 2015, from 9-Excellent Condition to 7-Good Condition. Bridges with better deck condition are more susceptible to decreasing in deck condition, and decrease in deck condition more quickly than bridges with poorer deck condition because the change from excellent to good condition describes far less deterioration than the change from good to fair condition.

Without proper assessment of both surface and subsurface deterioration, and without more accurate reporting of bridge deck condition rating, the rate of deterioration cannot be estimated. It is therefore much harder to detect deterioration in its early stages when smaller and more continuous improvement efforts can be made. When surface or subsurface deterioration continues unnoticed, it can ultimately compromise structural integrity, and the comfort, cost, and most importantly safety of the public.

6.2 Recommendations

Upon completion of this study, it is determined that reported bridge deck condition from routine bridge inspection may be misleading. From simple visual inspection, it was estimated that the three bridges should not have the same deck condition rating in 2015, as the best condition bridge had isolated hairline cracks while the worst condition bridge has significant potholes with exposed rebar.

This was further verified after completion of surface roughness testing indicated that the IRI value for the Potowomut Bridge was nearly double that of the Ramp BB Bridge and four times that of Major Nathanael Greene Bridge, yet all three bridges had a deck condition rating of 7-Good Condition as reported by NBI in 2015. Also, completion of GPR testing over time, from 2015 to 2017, indicated that the subsurface conditions for the Potowomut Bridge worsened nearly 1.5 times more than that of the Ramp BB Bridge, and more than three times that of the Major Nathanael Greene Bridge, yet all three bridges remained unchanged in deck condition rating over the two-year time period.

Though it is easier to decrease in deck condition rating from a 9 to a 7 as Major Nathanael Greene did prior to this study, than it is to decrease from 7 to 6, the effects of the change in deterioration are not as significant for structural integrity or for motorists. With these surface and subsurface findings, it is evident that substantial deterioration of a concrete bridge deck can be missed during routine inspection when advanced technologies are not implemented.

Currently, visual and simplistic methods for assessing concrete bridge deterioration do not provide as much detail as do advanced technologies. Surface roughness testing to aid in the assessment of deck surface deterioration provides better mapping of deck surface deficiencies and produces the IRI value which quantifies perceived roughness. Both maps and IRI values can be compared to previously recorded condition to monitor which areas are deteriorating and to what extent when testing is performed every two years like routine bridge inspection. Ground penetrating radar testing to aid in the assessment of deck subsurface deterioration reveals what is often hidden from the naked eye and crude basic testing. If mapped every two years like routine bridge inspection, there is no need for costly and time-consuming corroborative testing methods or deterioration thresholds, as comparisons can be made to previously recorded condition to determine which areas are deteriorating and to what extent.

By comparing current subsurface condition to previously recorded condition, it is determined that concrete bridge deck inspection can certainly be improved upon, and that deck condition rating may be underreported, as all three bridges should obviously not have the same deck condition rating in their current conditions. Deck condition rating may be misleadingly reported because advanced technologies are not being implemented to better assess both surface and subsurface deterioration. It is therefore recommended that both surface roughness testing and ground penetrating radar testing be performed during routine bridge deck inspection. When possible, testing using these advanced technologies should be performed upon initial bridge deck construction, to obtain a baseline for sound conditions free from deterioration. Using this baseline, change in condition over time can be more easily analyzed.

A fuller picture of both surface and subsurface deterioration obtained using advanced technologies allows for better evaluation of overall deterioration in concrete bridge decks. A more thorough assessment of overall bridge deck deterioration and change in bridge deck condition over time leads to more accurate reporting and monitoring. This translates to management agencies being able to make smaller, more continuous, and more cost effective improvement efforts in the present, rather than major replacement or rehabilitation efforts in the future. In making improvement efforts at the most optimal time, before substantial deterioration occurs, the structural integrity of the bridge deck, as well as the comfort, cost, and most importantly safety of the public is preserved.

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APPENDICES

APPENDIX A

A.1: SurPro Equipment Procedure and Settings

Preparing the Profiler

1. Together with helper, using proper lifting technique, remove case from vehicle and place on safe, level area
2. Release latches and open case lid
3. Remove kickstand and place on safe, level ground
4. Using proper lifting technique, lift out base unit and place on kickstand
5. Remove handle and attach to base using the two thumbscrews
6. Remove control cabinet and attach to handle using swivel bracket
7. Connect control cabinet cable to base unit, wrapping around the handle shaft twice to eliminate loose cable
8. Inspect all connectors and hardware for tightness and damage

Prior to Profiling

1. Check that both the USB in unit, as well as backup USB, are both empty
2. Ensure that the battery is fully charged (drain battery down to 11V before recharging; plug charger into unit before connecting to outlet)
3. Remember to bring the two USBs, a tape measure, a 300 foot long tape, multiple cans of spray paint, the battery charger, and safety precautions out into the field
4. Turn power on (flip switch surrounded in red located on the front of the base unit near arrow)
5. Let unit stand for 15-20 minutes to adjust to testing environment
6. Check shocks on base unit for fluid motion and ensure that all springs are properly aligned
7. Press the "MENU" button, then press the "YES" button to select "1. Data & Controls" to set unit key parameters. Make sure:
    a. "1.C01 Data & Control System Units?" is set to feet
    b. "1.C24 Data & Control Wheel Spacing (ft)?" is set to 0.82021
    c. "1.C38 Data & Control Sample Dist. (in)?" is set to 1
8. Using the tape measure, 300 foot long tape, and spray paint, lay out a two foot by two foot grid across the length and width of the entire bridge (or what part is applicable for testing)
9. Run unit along test line (forward and reverse) to allow unit tires to adjust to testing environment

10. Ensure that all safety precautions are accounted for (traffic control, proper safety attire, etc.)

Calibration (perform daily)

Check Distance Calibration

1. Press the “RECALL” button on the control cabinet
2. Using the arrow buttons, press the down arrow button until page 8 is reached (until the bottom of the display screen reads “Recall Pg. 8/18”)
3. Ensure that the “en_dist_cal” value is approximately 104,432.377 or within the range 96,399-113,926 pulses/meter
4. Make a profile run without saving: after pressing the “STOP” button, ensure that the measured distance value is within ¼ of an inch from the previously measured 150 feet

Elevation Calibration

1. Press the “MENU” button and use arrows to navigate to “9. Pick Operate Mode”.
2. Press the “YES” button, then select “A. Normal Rolling” as the operating mode of choice using the arrows and “YES” button
3. Perform a closed loop:
   a. Make a standard profile run for forward run without saving
   b. After pressing the “STOP” button, turn the unit around and press the “REV” button, then continue with usual data collection procedure for reverse run
4. After stopping, when asked “Save New Cal? YES/NO” press the “YES” button to save the new calibration

Making a Profile Run

1. Align the middle of the unit over the start of the first grid line longitudinally
2. Press the “CLR” button when the distance value in the upper right hand corner reads exactly 0.00 feet, and then press the “RUN” button
3. Push the unit as straight and without tilt as possible along the grid line, maintaining an approximate speed of 1.25-2.5 MPH, along the entire length of the line. Stop pushing when the middle of the unit has reached the very end of the grid line
4. Press the “STOP” button to stop collecting data
5. Press the “RECALL” button to recover the most recent run file information. The arrow buttons may be used to scroll through the various recall data screens
6. Press the “SAVE” button to save the data to the onboard solid state drive
7. Press the “SEND” button to send/download the data using USB port to flash drive. Select option “A. Send Current File” using the arrow buttons and the “YES” button

*While making a profile run, press the “EVNT” button to record an event if sources of error are encountered, so that “flags” are present in generated ProVAL graphs as markers

A.2: GPR Equipment Procedure and Settings

Before Testing

- ☐ Charge both batteries/Pack charger with power cord and extra batteries
- ☐ Check battery status (TerraSIRch → System → Battery → Status)
- ☐ Make sure SIR-3000 storage space is available/Pack USB external drive
- ☐ Pack measuring tape, spray paint, paint wand
- ☐ Pack clipboard, bridge info sheet, field testing sheet, camera, etc.
- ☐ Pack transit/target prism survey equipment
- ☐ Note rebar information (top rebar direction, rebar size and spacing, rebar type, etc.)
- ☐ Configure SIR-3000 (see settings below)

During Testing

- ☐ Turn off all cell phones
- ☐ Unload GPR survey cart and check antenna is secured in the bottom white tray
- ☐ Connect USB to controller **BEFORE** turning unit on
- ☐ **Layout grid (2'x2')**
    - Note start location (use bridge curb as a reference)
    - Record distance of start curb location to an absolute reference point that can be associated with a bridge drawing. These points include the bridge railing, drainage grates, and the side of the bridge.
    - Note scanning direction (perpendicular to orientation of top bar)
- ☐ **Determine skew angle of bridge**
    - Set the transit up on the intersection of the bridge joint and the edge of pavement
    - Place the target rod on the same edge of pavement as the transit down the bridge, far enough away to target the prism
    - Target the prism and set the angle to zero (**Note: This option is only available once the transit has been properly leveled.**)
    - Place the target rod on the same bridge joint as the transit across the bridge
    - Target the prism and record the horizontal angle
- ☐ **Calibrate distance measuring instrument (DMI)**
    - Collect → Radar → Mode
    - Switch setting off of Distance and then reset to Distance to open Distance Calibration Window
    - Input the desired calibration distance
    - Follow the on-screen guide to complete calibration
- ☐ **Set gain**
    - Collect → Scan → Auto (Points = 5; System will automatically set proper gain)
    - Write down the gain values
    - Collect → Scan → Manual (system will lock in the number of points and gain for the entire test)
    - If you change batteries or need to restart system, **RE-ENTER GAIN VALUES** (Collect→Scan→Manual)
- ☐ Turn on antenna (Press Run/Stop, green light below "Mark" should be green)
- ☐ Place cart before bridge joint. Press Run/Setup to start and stop recording data. After 3 beeps, start moving cart. Collect data beyond the end of bridge.
- ☐ Save file
- ☐ Zig-zag along grid

After Testing

- ☐ If data was stored on internal memory, transfer to USB
    - OUTPUT → TRANSFER → HD →Select files

- If the USB was connected to the controller before turning the system on, then this step is not necessary.

☐ **Check USB drive on computer to make sure that all files were transferred before deleting**

☐ Delete copied files from the GSSI internal memory (internal memory is only 1GB)

- OUTPUT → TRANSFER → DELETE →Select files

☐ Scan field notes and Caption all photos

GPR SIR-3000 Configuration

SYSTEM	COLLECT		PLAYBACK
Units: DEPTH = inches DISTANCE = foot VSCALE = time **Setup:**[1] Recall Save Path = Common Backlight = 4 Date/Time: check Battery: check Language: English	**Radar:** 1.5/1.6 GHz T_RATE[2] = 100kHz MODE[3] = Distance GPS = none **Scan:** Samples[4] = 512 Format = 16 bit Range[5] = 15 ns Diel[6] = 7.5 Rate[7] = 120 Scan/Unit[8] =120 Gain[9] = 6 db	**Gain:** Auto/Manual[10] Points GP1-5 **Position:** Auto[11] Offset[12]: Do Not Change Surface[13] = 10% **Filters:**[14] LP_IIR = 0 HP_IIR = 10 LP_FIR = 3000 HP_FIR = 250 Stacking = 0 BGR_RMVL = 0	Scan:[15] Diel/Surface Process: [16] **OUTPUT** Display: [17] Transfer: [18] **DATA COLLECTION** 1. Connect USB before turning unit on 2. Recall/setup parameters 3. Calibrate DMI (see below) 4. Set Gain (Auto then Manual to lock in values) 5. Press Run/Stop to turn on antenna 6. Press Run/Setup to start/stop recording data after 3 beeps, start rolling cart 7. Save file

(1) allows you to save/recall data collection parameters
(2) transmission rate capped at 100 KHz
(3) distance based collection using DMI (scans/ft)
(4) samples/scan, as sample number increases, max. scan rate decreases and file size increases (512 or 1024 recommended)
(5) two-way time window that system will record reflections (10-15 ns is recommended for 1.6GHZ antenna on concrete bridge decks)
(6) approximate setting of the dielectric constant of material which reflects the velocity of EM wave through a materiel, higher values mean slower travel time and shallower penetration, asphalt (3-5), concrete (5-8); not critical for deterioration mapping
(7) number of scans system will record in RAM per second. If set too high, system will automatically lower it to max. possible
(8) scans/ft, 120 scans/ft = 10 scans/inch
(9) display gain; this is not saved in data file
(10) initially set to Auto to get values, then turn to Manual and input same values to lock them in
(11) controls the position of time-zero
(12) this is a system parameter, do not change. Describes the time lag (ns) from the controller triggering the pulse until the signal is transmitted from the antenna; direct coupling is the pulse that travels inside the antenna housing directly from the transmitter to the receiver; use the direct wave to locate ground surface
(13) display option that sets a percentage of the data to display; system will set ground level near first positive peak of the direct coupling
(14) set automatically based on the antenna used; LP/HP = low pass/HP; Infinite Impulse Response (IIR) and Finite Impulse Response (FIR)
(15) diel/surface options are duplicated here; they are the same as in "Collect"
(16) allow you to change filters; setting doesn't permanently alter data; only for display purposes
(17) change "look and feel" of data displayed on screen
(18) data transfer from internal memory to Flash or HD (hard drive)
(19) in setup mode-starts/stops antenna; in run mode-stops data collection and brings up crosshairs, push again to bring up save file window

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