GEORGIA DOT RESEARCH PROJECT 17-05

FINAL REPORT

QUANTIFYING THE IMPACT OF COVER DEFICIENCIES ON BRIDGE DECK SERVICE LIFE: RECOMMENDATIONS FOR CONTRACTING

Georgia Department of Transportation

OFFICE OF PERFORMANCE-BASED MANAGEMENT AND RESEARCH

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Construction of reinforced concrete bridge decks with shallower or deeper-than-specified concrete cover remains an ongoing challenge for the Georgia Department of Transportation (GDOT) because it has the potential to lead to reductions in service life. This reduction in service life can result in negative consequences on traffic (and create an associated economic impact), in addition to reducing ride quality and bridge safety. To mitigate these factors, the overarching goal of this research project was to explore the use of scientifically-based contracting mechanisms as a means to prolong bridge deck service life. Specifically, the research objectives were (1) to understand the extent of cover depth variability in Georgia through interviews with construction and maintenance engineers at GDOT, (2) to use that data to model service life for various cover depths, concrete mixtures, reinforcement types, extent of cracking, and environmental conditions, considering the range in materials and exposure conditions in the state, (3) to examine contracting arrangements used in other states, (4) to assess what portion of construction costs are reasonably associated with service life, making recommendations for an appropriate liquidated damage for time delays or other performance penalty structures for less-than-specification cover depths, including a lower bound where reconstruction or immediate amelioration should be undertaken; incentives for contractor performance were also to be considered, and (5) to draft recommended practices for GDOT for contracts for bridge deck construction based upon performance.
Through interviews and an analysis of historical records, this project examined concrete cover practices and variability. The observed cover ranges were then related to the long-term durability of the bridge deck through corrosion-based service life modeling. The results of the modeling, in combination with current GDOT specifications, were then used to explore alternative contracting methods that could be adopted such as adjustable payment plans, for the purpose of incentivizing better cover control in new construction.</td></tr>
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GDOT Research Project 17-05

Final Report

QUANTIFYING THE IMPACT OF COVER DEFICIENCIES ON BRIDGE DECK SERVICE LIFE: RECOMMENDATIONS FOR CONTRACTING

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In cooperation with
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LIST OF ABBREVIATIONS AND SYMBOLS

AASHTO	American Assocation of State Highway and Transportation Officials
AC	Average cover
ACI	American Concrete Institute
AQL	Acceptable quality limit
ASTM	American Society for Testing and Materials
C	Chloride concentration
C_s	Surface chloride concentration
C_t	Chloride threshold
C_0	Initial chloride concentration in concrete
C_1	Chloride concentration at the surface (C_s equivalent)
CDF	Cumulative distribution function
D	Chloride diffusivity in concrete
DC	Design cover
E	Maximum acceptable deviation between the true and sample averages
erf	Error function
FHWA	Federal Highway Administration
GAMS	Georgia Asset Management System
GDOT	Georgia Department of Transportation
LSL	Lower specification limit
N	Number of samples per test lot
NIST	National Institute of Standards and Testing
NBI	National Bridge Inventory
OMAT	Office of Materials and Testing
PF	Pay factor
pH	Potential of hydrogen
PWL	Percent within limits
RQL	Rejection quality level
SOP	Standard operating procedure
T	Time
t_i	Corrosion initiation period
t_p	Corrosion propagation period
t_{sl}	Service life of a bridge deck
TxDOT	Texas Department of Transportation
USL	Upper specification limit
VTrans	Virginia Agency of Transportation

x	Concrete cover
ζ	Standard deviation of a lognormal distribution
λ	Mean of a lognormal distribution
μ	Mean
σ	Standard deviation
σ_{center}	Standard deviation of the results of a process about a target value
$\sigma_{combined}$	Standard deviation accounting for process and center deviations
$\sigma_{process}$	Inherent standard deviation of a process
σ_0	A priori estimate of the standard deviation for a process
φ	The concentration at distance and time

EXECUTIVE SUMMARY

Construction of reinforced concrete bridge decks with shallower or deeper-than-specified concrete cover remains an ongoing challenge for the Georgia Department of Transportation (GDOT) because it has the potential to lead to reductions in service life. This reduction in service life can result in negative consequences on traffic (and create an associated economic impact), in addition to reducing ride quality and bridge safety. Through interviews and an analysis of historical records, this project first examined concrete cover practices and variablity. Observed cover ranges were then related to the long term durability of the bridge deck through corrosion-based service life modeling. For the purpose of incentivizing better cover control in new construction, the results of the modeling, in combination with current GDOT specifications, were then used to explore alternative contracting methods that could be adopted such as adjustable payment plans.

1. INTRODUCTION AND BACKGROUND

1.1 Introduction

Bridges are critical structures, serving an important function vital to the safe and economical conveyance of people and goods throughout Georgia. However, construction of reinforced concrete bridge decks with shallower or deeper-than-specified concrete cover remains an ongoing challenge for the Georgia Department of Transportation (GDOT) because it has the potential to lead to reductions in service life. Shallow cover shortens service life in bridge decks through earlier incidence of corrosion and more rapid degradation during corrosion. Conversely, cover depth that is too deep also leads to deck cracking, contributing to degradation through corrosion, salt scaling, and/or freeze/thaw cycling. Premature degradation necessitates for more frequent inspections, earlier and additional maintenance and repair, and eventually, reconstruction. These activities have negative consequences on traffic (and create an associated economic impact), in addition to reducing ride quality and bridge safety.

Currently, no formal financial incentives or consequences exist for contractors and associated parties that construct a bridge deck that does not meet the design cover requirements dictated in the Georgia concrete bridge deck specification. With GDOT anticipated to contract placement of 150 to 200 bridge decks in the coming years, improvements in the contracting for this construction are a current and vital topic. The overarching goal of this research project was to explore the use of scientifically-based contracting mechanisms as a means to prolong bridge deck service life. Specifically, the research objectives were:

1) To understand the extent of cover depth variability in Georgia through interviews with construction and maintenance engineers at GDOT.
2) To use that data to model service life for various cover depths, concrete mixtures, reinforcement types, extent of cracking, and environmental conditions, considering the range in materials and exposure conditions in the state.
3) To examine contracting arrangements used in other states.
4) To assess what portion of construction costs are reasonably associated with service life and make recommendations for an appropriate liquidated damage regarding time delays or other performance penalty structures for less-than-specification cover depths, including a lower bound where reconstruction or immediate amelioration should be undertaken; incentives for contractor performance were also to be considered.
5) To draft recommended practices for GDOT bridge deck construction contracts based upon performance.

The research objectives in this project were addressed by a multi-stage approach. The first stage featured analyzing historical construction and inspection records, as well as performing interviews with GDOT personnel. This was done to determine the current construction practices and expected bridge deck performance. The cover surveys and biannual inspection reports from in service and recently decomissioned bridges were then analyzed to determine the expected construction variability as well as the predominant degradation mechanisms. Based on these insights, the second stage involved creating service life models with various methodologies and level of complexity in order to predict the performance of bridge decks created with different concrete cover distributions under variable material and environmental conditions. The final stage used the findings of the previous two stages to select and evaluate different contracting methods that may be used in future construction to achieve better cover control.

1.2 Report Organization

Chapter 2 of this report discusses GDOT cover practices and variability. The chapter summarizes the findings from interviews, field observations, and historical records.

Chapter 3 describes the cover surveys and biannual inspection reports, and then summarizes their findings in terms of predominant bridge degredation mechanisms.

Chapter 4 contains the service life modeling. Specifically, this chapter provides both simplified and complex models that can be used to predict the performance of bridge decks with insufficient covers. This chapter also contains a discussion on other methods that could be used to increase service life, such as considerations for concrete quality.

Chapter 5 contains a review of current contracting methods from other states and presents contracting methods that GDOT could apply to construction with inaccurate cover.

Chapter 6 explains the research project’s conclusions.

Chapter 7 lists the references cited in this report.

The Appendices contain select cover surveys, the GDOT mix design guidelines, select raw data from decommisioned bridges with deck defects, and draft standard operating procedure.

2. GDOT CONCRETE COVER PRACTICES AND VARIABILITY

This chapter contains information on the practices and variability of GDOT bridge deck cover. Specifically, Section 2.1 discusses the findings and observations from interviews and site visits as they pertain to construction methods and cover measurements. Section 2.2 presents the findings from the analysis of historical cover surveys. The analysis quantifies the cover variability of bridges since 1970 and provides probability functions that are used throughout the research.

2.1 Current GDOT Cover Control Methods – Field Interviews and Observations

In order to establish a practical understanding of the techniques and methods used to ensure appropriate deck concrete cover in Georgia, the research team attended the concrete pour of an I-85 interstate bridge in April 2018, shown in Figure 1. The site was located one mile south of Exit 115 (SR 20) on I-85 south, about 38 miles to the northeast of Atlanta. The design of this section included epoxy-coated top rebar mat and a specified cover of 2.75 inches. During the construction, researchers interviewed GDOT, Federal Highway Administration (FHWA), ATKINS, and C.W. Matthews personnel present.

Figure 1. Photo. I-85 Exit 115 construction prior to concrete pour.

During the field visit, the procedure for verifying correct concrete cover was observed as a multi-stage process utilizing three methods at three times in the construction process: during the “dry run”, during the pour, and on the cured concrete, about a month following the pour. This process is summarized in Figure 2 and is discussed in detail in the following sections.

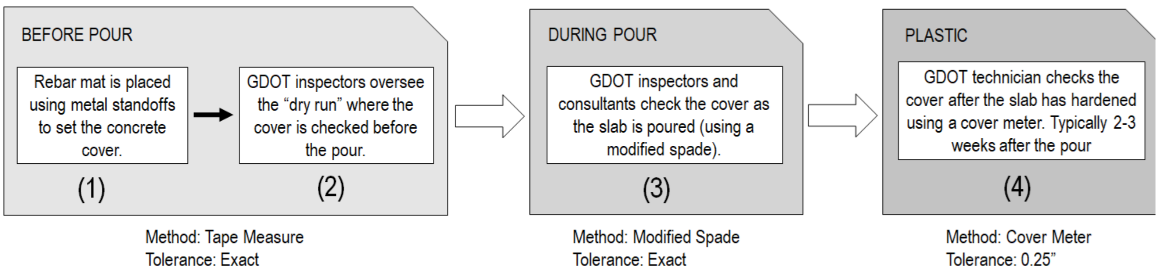


Figure 2. Process. Summary of cover measurements during construction.

2.1.1 Cover Measurement – Before Pour

First, the elevation of the rails that the finishing machine rides on is set (one on each side along the length of the deck), which will ensure that the bottom of the screed is at the correct height along the deck. The rails must be level and straight, as they serve as the reference for all reinforcement placement. Key elements of the cover verification process are shown in Figure 3. If the rails are incorrectly set, the resulting cover could be placed systematically too shallow or deep.

Figure 3. Photo. Key elements of the cover verification process.

The next step is to place the lower standoffs, called chairs (see Figure 4), which separates the lower rebar mat from the bottom of the slab (top of the stay-in-place formwork). The lower mat bars are placed and then tied together with rebar ties. Next, larger chairs are placed in order to support the upper rebar mat. The upper mat rebar is then put into place and tied together.

Figure 4. Photo. Chairs used to set appropriate standoff distance between rebar mats.

Next, the contractor, under the supervision of GDOT and the consultant for the project, performs a “dry run” of the pour. Typically the dry run occurs the day before the actual pour. During the dry run, the screed is moved systematically over the deck (side-to-side and then front-to-back), with cover measured laterally every ten feet, and longitudinally at least once per deck bay. To measure cover, the screed head is fitted with a leveled piece of lumber as shown in Figure 5. The height of the lumber represents the future top of the deck. A GDOT inspector measures the perpendicular distance from the top of the rebar mat to the bottom of the lumber with a measuring tape. Discussions with the GDOT personnel indicated that the cover measurements need to be “exactly” correct at this point. Any deviations from the expected values are fixed by the contractor on the spot.

Figure 5. Photo. GDOT inspector measuring the concrete cover using a tape measure and lumber affixed to the bottom of the screed head.

2.1.2 Cover Measurement – During Pour

Following the approval of the deck dry run and a “prepour conference” where any remaining issues are discussed and addressed, the deck is ready to be poured. During the pour, GDOT inspectors and hired contractors verify the cover, along with the slump of the concrete, air entrainment, and pour rate. At this stage, inspectors use a modified shovel to measure the concrete cover. The modified shovel, shown in Figure 6, has notches in the blade portion at both 2.5 inches and 2.75 inches from the edge of the blade, as well as an 8.25 inch-long spike on the opposite side for measuring total slab thickness. To perform a measurement, the inspector plunges the blade into the fresh concrete (typically within a minute after the screed has passed) until it makes contact with the top mat. Then the inspectors pulls the blade out. If the residual concrete does not reach exactly the level of the premade notch, the inspector measures the residual level on the blade using a tape measure.

Figure 6. Photo. Modified shovel used to measure concrete cover consisting of a blade end and a spiked end. An expanded view of the blade shows the right-side notch, which is located 2.5 inch from the blade's edge.

2.1.3 Cover Measurement – Plastic Concrete

From discussions with GDOT personnel, the measured cover of the finished deck should be within 0.25 inch of the design-specified cover. About a month after the pour, GDOT personnel from the Office of Materials and Testing (OMAT) assess the deck roughness and verify cover using a cover meter. This step serves as the last step in the process and ensures that the finished deck has the correct concrete cover.

It is important to acknowledge the accuracy of such cover measurements taken during the final check, with two primary areas of concern: the cover survey sampling grid size (e.g., measured in a 10 foot by 10 foot grid) and the cover meter accurcy. When discussing the grid size, it is first useful to examine the requirements in GDOT specifications. Specificially, the Bridge, Culvert, and Retaining Wall Construction Manual (GDOT 1996) includes the following language and Figure 7 to specify the grid size for deck depth checks:

> *"Deck depth checks should be made over the beams or supports and midway between beams or supports in a sufficient number of locations to reasonably be assued that plan bar reinforcment cover and slab thickness is obtained (GDOT 1996)."*

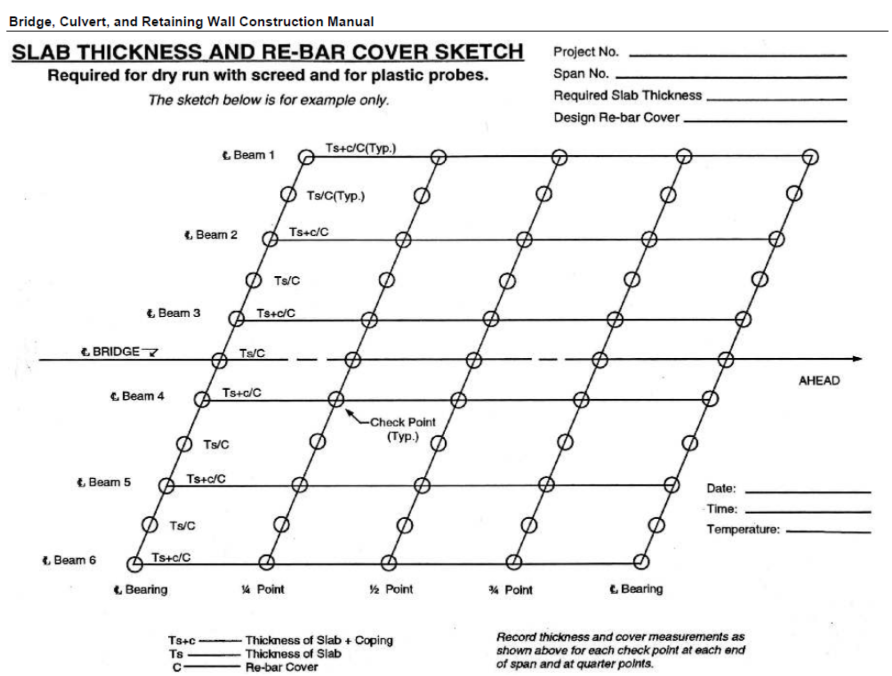


Figure 7. Illustration. Required cover survey grid (GDOT 1996).

From Figure 7, it appears that the grid is highly dependent on the span length and girder spacing. For a bridge with 100 foot span, Figure 7 would only require a cover measurement every 25 feet along the length. For a span as short as 30 feet, a measurement would be required every 7.5 feet along the span. However, after consulting with GDOT personnel, it appears that the cover is typically measured within each girder bay, and approximately every 10 feet along the span regardless of span length. If the cover control is in question, more cover measurements are taken. The 10 foot increment appears to be based on the experience and judgement of GDOT personnel with the guiding standard of being "reasonably assured" of appropriate cover control. It may prove, however, that a smaller grid size is beneficial for more accurate service life modeling, as significant local variations in cover may be possible within 10 feet of span length.

Cover meter accuracy is also evaluated in a paper by Barnes and Zheng (Barnes and Zheng 2008). The goal of the paper was to determine whether cover meters, under a variety of conditions, are accurate to the +/- 5% or +/- 0.08 inch specification prescribed in British Standard 1881-204 (British Standards Institutions 1988). The researchers examined the accuracy of two different commercial cover meters on the following parameters: depth to the rebar, distance between the rebar being measured and neighboring secondary bars, and sensitivity to selecting the wrong bar diameter setting on the meter. The researchers found that, under normal circumstances (correct bar diameter, reasonable cover depth, normal bar spacing), the devices performed well within the prescribed +/- 5% or +/- 0.08 inch accuracy. In their discussion, they noted that the maximum measureable cover depth decreases as the bar spacing decreases, that there was moderate sensitivity to using the wrong bar diameter, and that the location of secondary bars seemed insignificant.

However, the specified +/- 5% is expected only in laboratory conditions. The researchers specify a field tolerance of +/- 15% or +/- 0.197 inch as being the expected accuracy on an average site. Bridge decks in Georgia typically have either 2.25 inches or 2.75 inches concrete cover. For those cover depths, the +/-5 mm limit governs, and thus the cover measured should be within +/- 0.188 inch. While this range needs further verification for the conditions in Georgia, it is important to acknowledge that there exists variability in the devices used to measure cover, which may be consequential to the findings of this report.

2.1.4 Explanations for Cover Measurement Issues

Interviews performed during the I-85 site visit with both GDOT personnel and the contractor, as well as later on with personnel from the Office of Materials and Testing (who perform the cover surveys) yielded the following explanations for cover measurements being outside the accepable ranges during some parts of the process:

1) Contractors may intentionally pour excessive cover in anticipation of significant surface grinding.
2) Failed formwork.
3) The screed rails may be improperly set during the beginning of the process.
4) The rebar near the headers, the wooden formwork that seperates the slabs in a multi-span deck, tends to have too much cover as it is unsupported and tends to sink.
5) General human error.

2.1.5 Remediation of Improper Cover

Should cover be found well outside expected values, the concrete can be washed out of the slab formwork entirely if uncured, or hydrodemolished down to the rebar depth for concrete that is already plastic. From discussions with GDOT personnel, this is a rare occurrence. In those cases, following the washing or hydrodemolishion, the source of the cover control failure is addressed and the slab/cover is repoured. For decks nearer to the prescribed tolerances, other options for amelioration include the addition of an epoxy or copolymer overlay over the cover deficient deck. For excess cover, the surface may be ground to remove the extra cover. Discretion is entirely left to the Engineer for selecting any of the above remedies or outright rejecting the work, and requiring a new deck be constructed instead.

2.2 Cover Depth Variability in Georgia

2.2.1 Cover Surveys

To ascertain the cover depth variability in Georgia, an analysis of a series of steel cover surveys was performed. Data, including final steel cover surveys, was obtained from archival records provided by the GDOT Office of Materials and Testing. The bridges in the records included those constructed from 1978 to 2018. From that data, the surveys of 103 bridges were randomly selected for analysis.

Each survey provides information on the total number of cover measurements taken per bridge, the cover values, the average cover, the standard deviations, and (for select bridges) a two-dimensional representation of the cover distribution. On occasion, when the observed cover is well outside of expectations, comments related to validation by coring or corrective action are also noted. Figure 8 presents the average cover for the 103 randomly sampled bridges. Because the design cover prescribed in the late 1970s to the present has varied considerably, a normalization of the average cover is given. The normalized value was determined by subtracting the design cover from the average cover measured. This form of normalization was selected to preserve the sense of scale of the measurements at a minimal cost to the accuracy of the plot. To visually differentiate the multiple bridges per year, bridges for the same year are slightly offset from each other along the horizontal axis. Figure 9 summarizes the data shown in Figure 8 by displaying the average deviation from the design cover.

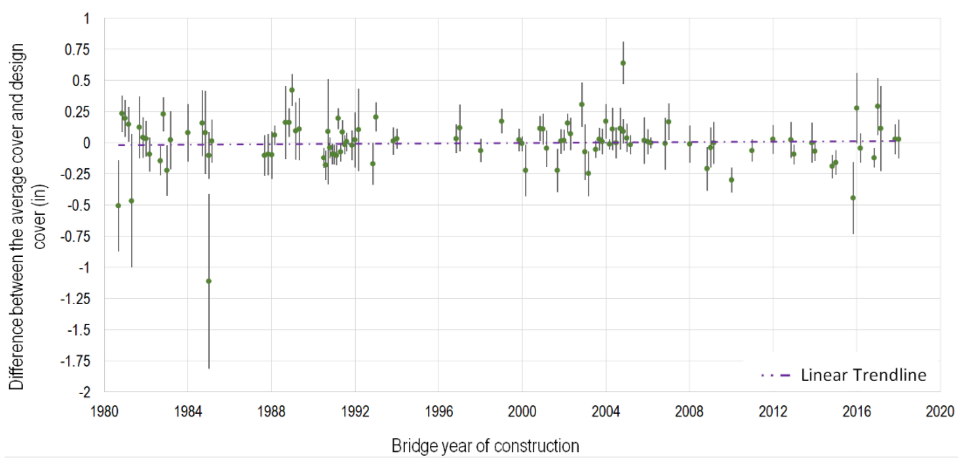


Figure 8. Graph. Normalized average cover for the 103 randomly sampled bridges with linear best-fit of the data.

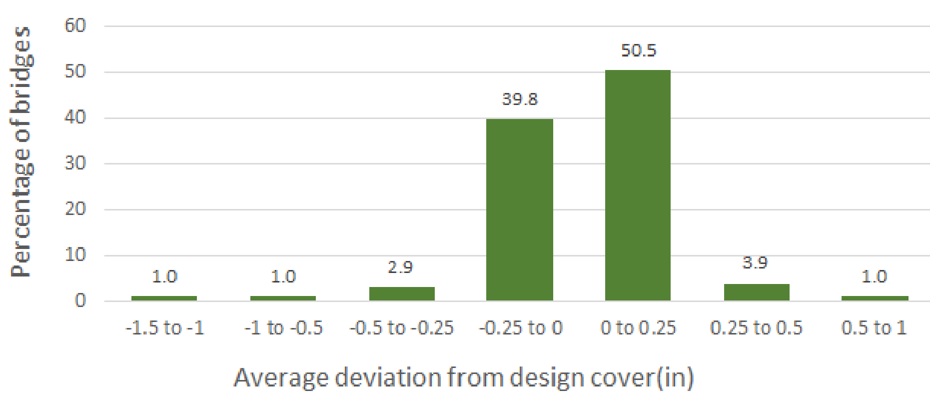


Figure 9. Graph. Average deviation from design cover for the 103 randomly sampled bridges.

The following general findings are noted from the results in Figure 8 and Figure 9:

1) There does not appear to be a trend towards excessive or insufficient cover, as shown by the flat-grade of the purple trendline in Figure 8.
2) Greater than 90% of bridges sampled have average covers within 0.25 inch of the design cover.
3) Even some modern construction bridges, such as the bridges built in 2016, have poor cover control.

It is important to note that the cover surveys are made pre-grind, and therefore represent the thickest the cover could be in service. Interestingly, the results in Figure 8 do not support the claims made during the interviews, namely that there often is excessive cover at this stage to accommodate subsequent grinding, which typically reduces the deck by up to 0.25 inch.

2.2.2 Two-dimensional Cover Maps

The previous section provided insights into the average cover per bridge, but did not take into account the cover as it varies across the deck. This section will focus on the variability and the distribution of the cover in the two-dimensional space (i.e., in the surface area of the deck). To investigate the two-dimensional distribution of cover, a secondary analysis was conducted using 11 auxiliary cover surveys pseudo-randomly selected from the 103 random bridges used before. The term "pseudo-randomly" was used because the procedure involved first randomly sampling from the full 103 bridge database, but then rejecting bridges that did not have two-dimensional cover information until 11 were selected. Each of the cover surveys from the 11 bridges are included in Appendix A.

The two-dimensional information was extracted from the surveys and plotted to form a surface map, divided into 16 colors. For each map, the range was set to +/- 1 inch from the design cover. The map linearly interpolates between the cover data represented by the vertices of the grid. The percentage of the deck area that is within each cover level (ex. 1.75 inches to 1.875 inches) was determined by software which divides the number of pixels for each cover level (determined by pixel color) by the total number of pixels.

Figure 10 to Figure 13 give examples of the two-dimensional cover maps. Each figure represents a different bridge that demonstrates a different type of cover map observed. Figure 10 shows the results from SR 72 bridge (P.I. No. 122100) and represents the best cover control as the cover is within a tight range, and there does not appear to be any areas with thin (red) or thick (blue) cover. Figure 11 represents a deck with a random and concentrated area of thin cover. The random nature suggests that a local issue with placing the rebar is likely the cause of the cover deviations. Figure 12 represents a deck with systemically thin cover, which suggests either a general misreading from the cover meter, poor rebar placement, or incorrect screed rail placement. Figure 13 also represents a deck with fully systemic poor cover. But, unlike in Figure 12, where areas of appropriate cover exist along the peripheries and in enclaves in the center, Figure 13 shows an almost totally insufficient cover distribution.

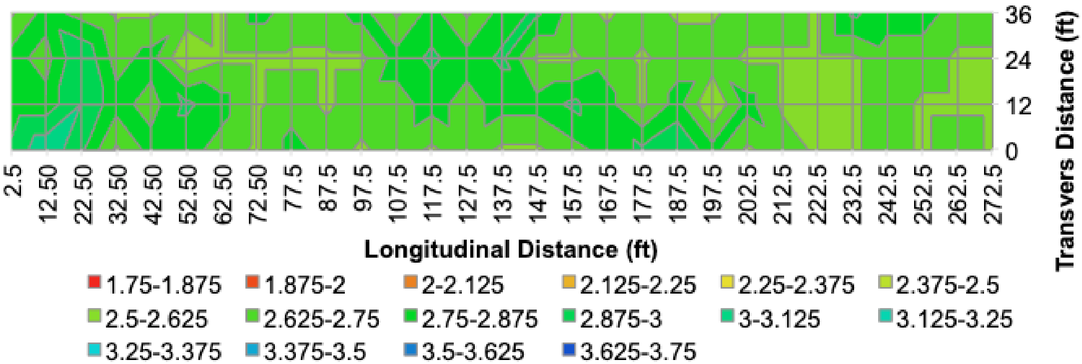


Figure 10. Map. SR 72 bridge cover map, which indicates good cover control throughout.

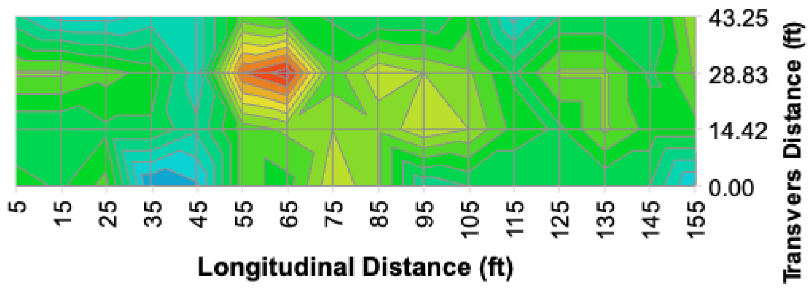


Figure 11. Map. SR 22 bridge cover map, which shows one occurrence of insufficient cover (red) and one major occurrence of excessive cover (blue).

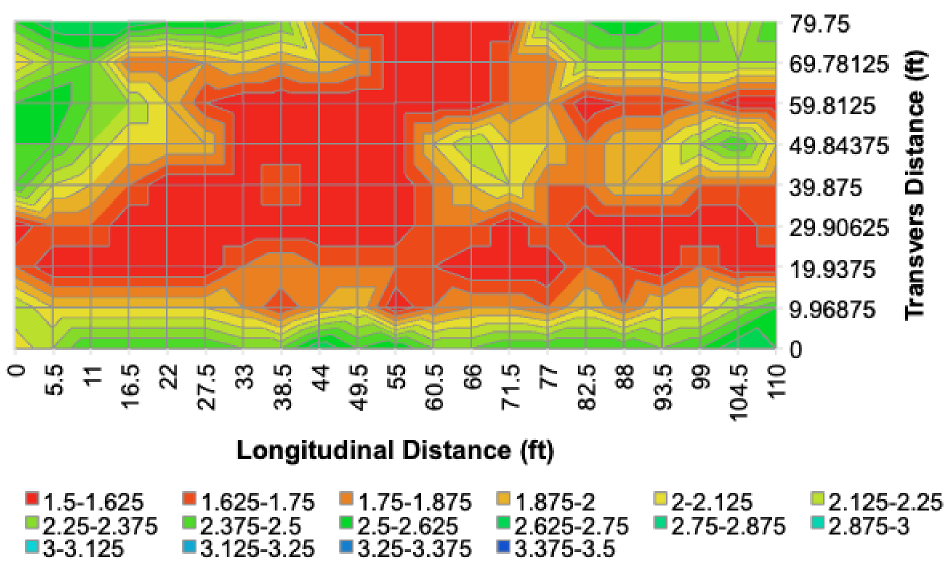


Figure 12. Map. I-85 bridge cover map, which shows systemic insufficient cover.

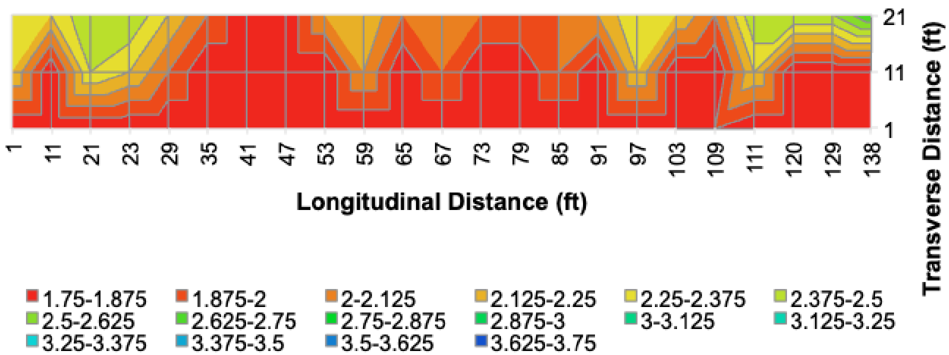


Figure 13. Map. Northlake Parkway bridge cover map, which shows systemic, but localized insufficient cover.

2.2.3 Cumulative Distribution Function

As described previously, the percentage of the deck area for each deviation was determined by software which uses the ratio of pixels for the color that designates each level to the total number of pixels. This procedure was performed on each of the 11 bridges independently. The results were then averaged into a single dataset. The purpose of this analysis is to determine the corresponding deficient deck area from numeric cover surveys. In real world applications the percentage of the deck that is deficient is more commonly used rather than the cover distributions, particularly when evaluating limit states. It is also extremely important when simulating a bridge deck for analysis to match not only the cover distribution but the corresponding surface areas. The results from this investigation are given in the form of a cumulative distribution function (CDF), as shown in Figure 14. A CDF was used as there may be interest in knowing the expected percentages below a certain threshold value for cover. From Figure 14, it appears that, on average, approximately 40% of the deck area is below design cover, and 60% is above design cover. It also appears that approximately 20% of deck area is more than 0.5 inch below design cover.

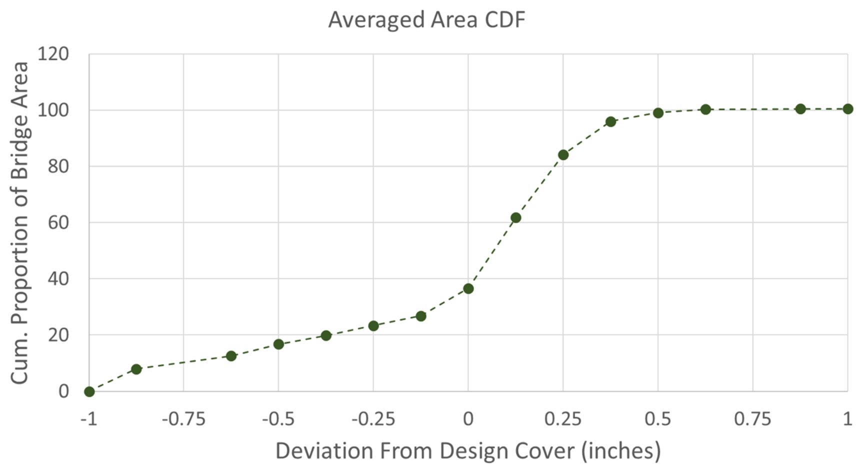


Figure 14. Graph. Cumulative Distribution Function (CDF) of bridge deck area per cover deviation.

2.2.4 Probability Distribution Fitting

In order to predict the cover distribution for bridges that we do not have the cover distributions for but are interested in modeling, a probability density function fit to the cover data is needed. Conventional wisdom is that the concrete cover distribution is best approximated by a lognormal distribution (Schießl, Bamforth et al. 2006). The rationale for this distribution is that the physical restraints (e.g., chairs) create a barrier preventing extremely insufficient cover, which would shift the probability distribution toward non-symetric forms. However, for a small range in the cover distribution, these external effects on the probabilities of the tails may not be important, and therefore a normal distribution may prove valid. To compare the different possible probability distributions fit for the observed cover distributions, and determine good default values, cover surveys were plotted in terms of cover versus the number of occurences of that cover measurement. Representative cover surveys are given in Figure 15. The form that the cover survey distributions have suggested that a lognormal or normal distribution may be suitable.

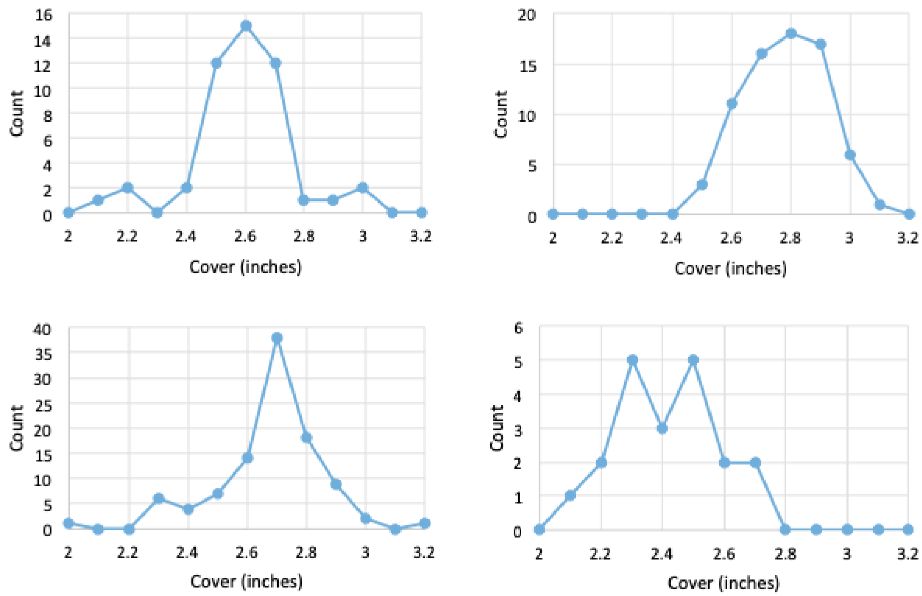


Figure 15. Graph. Representative cover survey distributions validating the use of lognormal or normal distributions.

A normal distribution, commonly refered to as gausian, is a symmetric distribution about its mean, μ, with a standard deviation, σ. In a normal distribution the probability of a value being twice as large as the mean is equal to the probability of being half as large. A lognormal distribution is a distribution of a random variable whose logarithim is normally distributed. Unlike the normal distribution, a lognormal distribution can only take positive real values, with a lognormal mean, λ, and a lognormal standard deviation, ζ. Having only positive real values and a natural skewness makes a lognormal distribution frequently used in multiple areas of engineering.

In order to determine what sampling method is best for our purposes, and what default values may be useful for modeling the cover distributions for bridges without that information, the two approaches were executed and compared:

- Approach 1: Randomly select a group of cover surveys and fit a lognormal and normal distribution to each survey's data separately. Calculate the average error for each distribution.
- Approach 2: Combine all cover surveys data together and fit a lognormal and normal distribution to the data. Calculate the average error for each distribution.

Using the first approach, 36 cover surveys with the same design cover (2.75 inches) were randomly selected and subjected to a least squared error fitting regimes. The results are presented in Table 1. Interestingly, from these results there does not appear to be a substantial difference in mean, standard deviation, and error between the fitted distributions, on average. The values suggest a natural variability to the cover distribution of about 0.11 inch, and an average cover approximately 0.1 inch below design cover. For this dataset, the error for the logormal distribution fit was approximately 8% less than that of the normal distribution fit.

Table 1. Summary of results from Approach 1.

Normal Distribution			**Lognormal Distribution**				
Mean, μ	Standard Deviation, σ	Error	Mean,μ	Standard Deviation, σ	Lambda, λ	Zeta, ζ	Error
2.68	0.11	30.6	2.68	0.12	0.98	0.04	28.0

Approach 2, which combines over 100 cover surveys from bridges with 2.75 inches of design cover yields an overall data set with an excess of 3700 individual cover measurements. It is important to note that some cover surveys were measured in 0.1 inch increments, while others were measured in 0.125 inch increments. The combined cover data are presented in Figure 16.

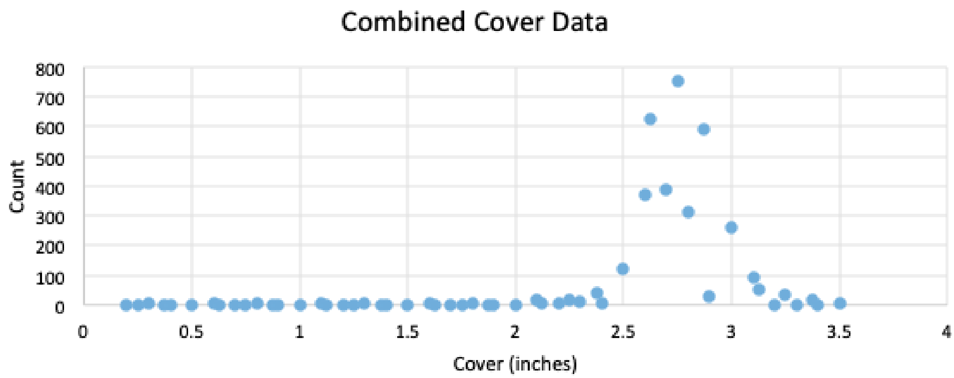


Figure 16. Graph. Combined data of over 100 bridge deck cover surveys.

The results from Approach 2 are summarized in Table 2. Consistent with the results from Approach 1, the results from Approach 2 show very similar performance between the fitting of a normal distribution and lognormal distribution to the data. The means are near identical to those from the prior evaluation, despite adding twice as many bridges as before. The standard deviations from this approach are approximately 33% larger than those from the prior testing, though they are very similar in absolute terms. The error for the lognormal distribution fit is less than that of the normal distribution fit, though perhaps not significantly so.

Table 2. Summary of results from Approach 2.

Normal Distribution			Lognormal Distribution				
Mean, μ	Standard Deviation, σ	Error	Mean,μ	Standard Deviation, σ	Lambda, λ	Zeta, ζ	Error
2.67	0.15	211429	2.67	0.15	0.98	0.06	210723

From this approach, an appropriate default set of mean and standard deviation to approximate Georgia bridge decks would be 2.67 inches average cover and 0.15 inch standard deviation, with equal suitability toward the use of a normal or lognormal distribution. These values do not significantly vary from the recommended 2.68 inches average cover and 0.11 inch average standard deviation from the results of Approach 1. For our purposes, a normal distribution with mean 2.68 inches and standard deviation 0.13 inch should be used to approximate the true cover distribtuion for a bridge built with a 2.75 inches design cover in Georgia.

3. QUANTIFYING BRIDGE DEGRADATION

A significant portion of the work undertaken in this project was to analyze and draw conclusions from all available inspection reports of inactive (also called decommissioned or deleted) bridges to determine causes that could have led to their decommissioning. This task was important in order to later determine which modeling strategies could be used for prediction and, further, which contracting mechanisms pertaining to cover are best to prevent such scenario.

3.1 Decommissioned Bridge Records

The Georgia Asset Management System (GAMS) was used to access the bridge reports used in this research. From GAMS, it was found that 524 bridges (including culvert structures) have been decommissioned since 2014. Excluding culvert structures, bridges without a concrete deck and/or bridges with reports that had significant information missing (such as year built, missing inspection reports, etc), the number of bridges available was reduced to 341. Decommissioned bridge reports prior to March 1st, 2014 are not available electronically through GAMS nor can they be found in paper form, though discussions with Mr. O'Daniels, Bridge Asset Manager for the GDOT Bridge Maintenance Unit, suggest that some portion of the older inspection reports may have been stored, and may be obtainable in the future.

For each bridge, the following general information was first extracted from the inspection reports: bridge serial number, latitude, longitude, year built, year replaced, year joints last sealed, service under type, service on type, and the National Bridge Inventory

(NBI) condition ratings. Table 3 proves a summary of each of the parameters and their definitions.

Table 3. Bridge insection parameters.

Parameter	Definition
Bridge Serial Number	The identifier used by maintenance personnel to identify each bridge in the inventory. The form is xxx-xxxx-x, with the first three values being the county number.
Latitude	The latitude coordinate of the bridge in degrees, minutes, seconds format
Longitude	The longitude coordinate of the bridge in degrees, minutes, seconds format
Year Built	The year when bridge construction completed
Year Replaced	If a bridge was replaced by another, the year that it was replaced
Service Under Type	The type of service that the bridge spans over, such as a waterway or a highway
Service on Type	The facility carried by the bridge, such as a highway or a country road
NBI Rating	The condition rating for each subcomponent of the bridge (deck, superstructure, substructure) on a 0-9 scale. The higher the NBI rating, the better the condition of that component.

3.2 National Bridge Inspection (NBI) Data

The National Bridge Inventory (NBI) condition ratings, created by the Federal Highway Administration, provide a quick evaluation of the overall condition of bridge components (NBI Coding Guide 1995). The NBI condition ratings are presented on a scale from zero to nine. Zero symbolizes a bridge in a failed condition while nine is indicative of excellent condition. From a discussion with GDOT personnel, it was stated that a bridge is typically replaced when the deck's NBI condition rating is a four. For reference, according to the NBI standards, an NBI rating of four is given when a component has "advanced section loss, deterioration, spalling or scour" (NBI Coding Guide 1995).

Each of the 341 decomissioned bridges from GAMS was analyzed using their NBI ratings. Figure 17 presents the ratings for the three major bridge components (superstructure, substructure, and deck). From the data, it appears that the most common rating for a bridge component was a six, a "satisfactory" rating. A satisfactory rating is given when "structural elements show some minor deterioration" (NBI Coding Guide 1995). On average, for a given bridge, the substructure had the lowest rating while the deck had the highest. It is important to note, however, that the relative ratings are in regard to the average, with individual bridges deviating from the trend.

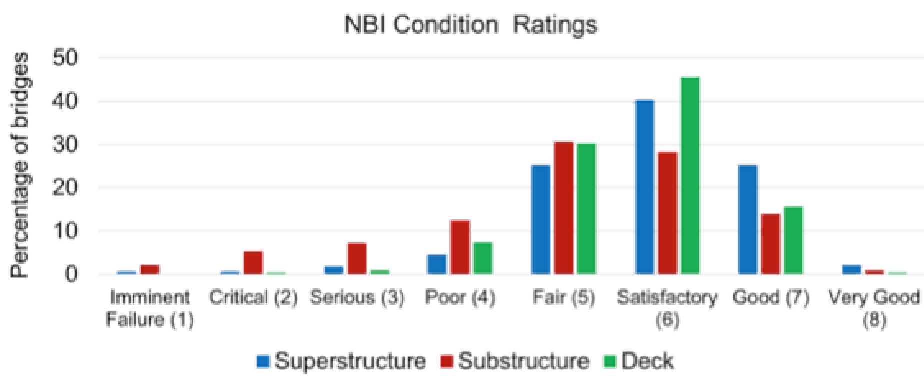


Figure 17. Graph. NBI condition ratings for the various components of the bridges at the time of decommissioning.

3.3 Conditions Contributing to Decommissioning

Selecting an appropriate degradation mechanism, which serves as the basis for any service life model, is vital in predicting the degradation a bridge deck experiences from its construction to the end of its service life. To inform that selection, it is prudent to examine the materials and construction of the bridge, as well as the intended service, and the environmental exposure. A poorly constructed bridge may degrade from the poor materials used or fail to withstand the stress imposed on it during its service life. A bridge in a rural environment may experience less wear on the road surface than an equivalent highway bridge, and thus abrasion may be less important in modeling the service life of a rural bridge. Harsh environments where chloride exposure is high may lead to bridges that degrade from corrosion, and so the service life model becomes a corrosion model. An analysis of decommissioned bridge decks aided in selecting an appropriate degradation mechanism for the work in Chapter 4.

3.3.1 Materials and Construction

The use of higher quality concrete mixes as well as alternative reinforcement (such as epoxy rebar) can have profound effects on the service life of the bridge. The higher quality concrete may better withstand abrasion and inhibit chloride ingress, two primary degradation mechanisms in bridge decks. Better construction practices, such as improved cover control may also increase the likelihood that a bridge meets its intended service life.

3.3.2 Service

The service conditions that a bridge experiences likely govern its degradation. Analysis from the bridge inspection reports showed that over 90% of the decommissioned bridges were highway bridges. Highway bridges experience significant cyclical car and truck traffic, which could lead to premature degradation when compared to an equivalent rural bridge.

3.3.3 Environmental

Equally important to the service on the bridge, is the environmental exposure. Performing an estimation of the number of waterway bridges in the total inventory shows that over 70% of the State’s bridges are over waterways. Therefore, the service under proportions are in rough alignment to the general population ratios, at least in terms of the percentage of waterway bridges. This is important because, depending on the salinity of the waterway traversed by the bridge, maritime bridges can face extremely corrosive conditions and thus are more susceptible to decommissioning.

3.4 Decommissioned Bridge Deficiencies

This section provides a summary of the deficiencies that decommissioned bridges had at the time of their decommissioning. The deficiencies were copied near verbatim from the inspection notes, without combining similar deficiencies, such as heavy and minor scaling. As expected, there is significant variability in terms of both the type and frequency of deficiencies for each of the major bridge components. Ranking the deficiencies in terms of total prevalence (summation of the prevalence in each major component) yields Figure 18.

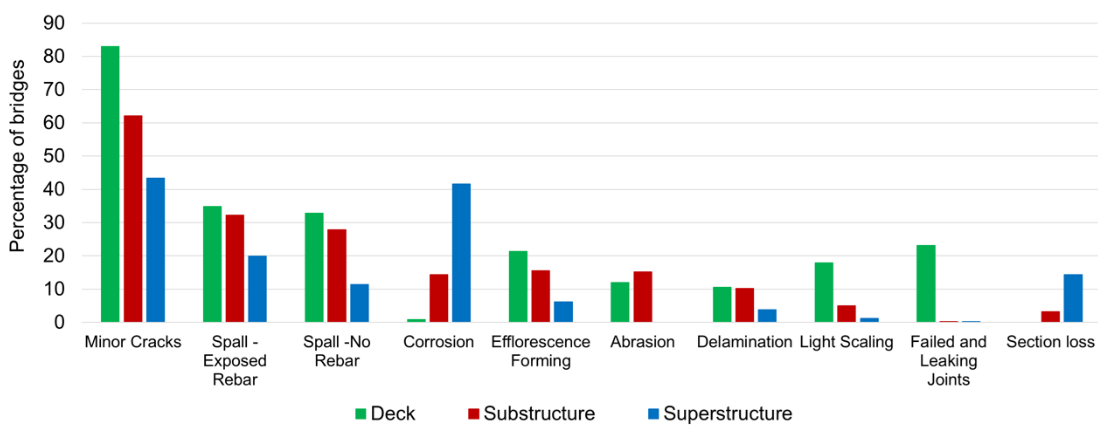


Figure 18. Graph. Top ten most prevalent bridge deficiencies ranked by total prevalence.

The data provide a comprehensive overview of the deficiencies noted (and those relatively absent) on the inspection reports. As alluded to earlier, certain conditions are interactive. For example, though no bridge inspection reports expressly described corrosion being present on/in the deck, concrete spalling is frequently the result of corrosion, and exposed rebar in all likelihood will corrode. Also, as noted previously, relative differentiation of the same deficiency are listed separately, as simply combining them into one deficiency may be misleading. With all those corollaries in mind, it appears that the primary deficiencies are the result of corrosion and mechanical wear. Minor cracking can be attributed to a variety of sources, such as thermal expansion and mechanical stress. The results in Figure 18 may also support the notion that a corrosion model may be required to adequately forecast the degradation of Georgia bridges.

Figure 19 shows the deficiencies ranked by prevalence in the decommissioned bridge decks. This figure further supports the notion that a corrosion model may be important in forecasting the service lives of Georgia bridges because the first eight symptoms are associated with corrosion, either by being caused by corrosion or being present in highly corrosive conditions. It is interesting that more than 60% of the decommissioned bridge decks had some form of spalling, which depending on the location and severity could severely disrupt the ability of traffic to safely pass. Also interesting is the prevalence of scaling, which may be the result of chloride exposure from de-icing activities. The presence of abrasion on a significant number of bridges (>10%) represents the other significant degradation mechanism: mechanical wear. On first approximation, it appears that neither a freeze-thaw model, a mechanical stress model, nor an alkali-silica reaction model will provide meaningful insights into the degradation of Georgia's bridges. Rather, a corrosion model, may prove best for the intended application, in alignment with past approaches and findings (Cady and Weyers 1983, Fanous and Wu 2000).

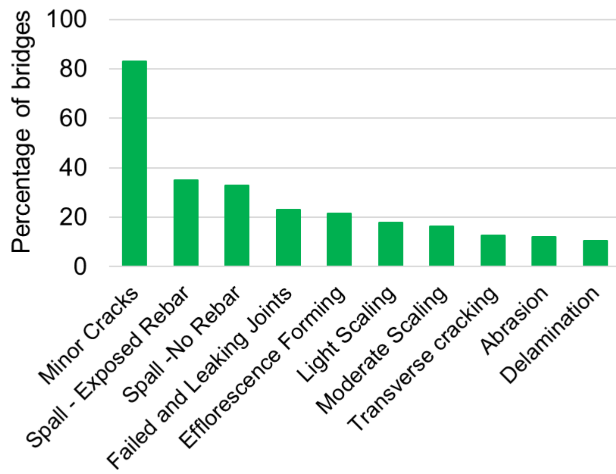


Figure 19. Graph. Deck deficiencies ranked by prevalence.

4. SERVICE LIFE MODELING

The overall approach developed for determining the effect that cover control variations have on the expected service life of a bridge deck was multimodal. The first approach used was cover threshold analysis (Section 4.1) to attempt to segregate the domain of cover distributions into areas, or thresholds, that have predictable performance (such as a threshold below which poor performance is guaranteed). This approach is the most basic, as it does not describe the mechanism of damage, and does not require large amounts of system information.

The second approach, discussed in Section 4.2, was a simple, one-dimensional corrosion analysis, whereby the service life of the deck is tied to the ingress of chlorides from the environment. Simple modeling has advantages over more sophisticated approaches when there is significant uncertainty in the modeling parameters, as is the case in this research. The one-dimensional model generates a rough estimate of the impact that variations in cover may have on the expected service life. Because it is important to consider the influence of variables aside from just the cover, such as the concrete quaility, a concurrent approach used commercially available corrosion service life models to examine the effects of concrete mix design variations on the expected service life.

The third approach, discussed in Section 4.3, developed and utilized a probabilistic simulation to expand on the conclusions of the other approaches, most notably the one-dimensional analysis. This method was chosen because of its affinity to uncertain systems. The probabilistic simulation is the most information intensive model but yielded more accurate predictions than more simple models.

4.1 Cover Threshold Analysis

It is important that the results from the modeling methods developed, as outlined in greater detail later in this section, are congruent to field performance. One simple, but effective, way to ascertain that is to separate the domain (i.e., cover distributions/deviations) into areas of similar performance. In other words, are there any cover thresholds below or above which damage is ensured or unlikely? To investigate that question, 50 of the 130 cover surveys were randomly sampled, and the final inspection reports from those bridges were gathered. The following information was aggregated from the reports: bridge age, design cover, average cover, standard deviation, and any bridge deck damage notations.

The results from the bridge inspection reports are presented in Figure 20 below, where the bridges that had damage are represented with a red triangle, and those without damage are represented with a green circle. For the ordinate, the deviation between the average and design cover was selected as that metric accommodates the large variations in design cover over the years. The abscissa is set to the bridge age, so as to illustrate whether the condition of the deck is predominantly dependent on age. The whisker for each point represents one standard deviation of the cover distribution, with larger whiskers representing less precise cover control.

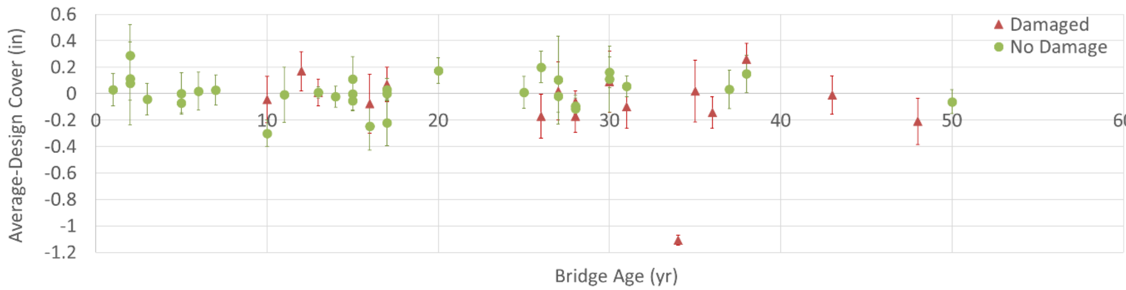


Figure 20. Graph. Comparison of the average deviation and bridge age to incidence of deck damage. The whiskers represent one standard deviation of the corresponding cover distribution.

From Figure 20, it is apparent that with exception of very large deviations in cover (in this case >1 inch), there does not appear to be a clear region where damage is near certain. Nor do there appear to be areas where damage-free performance is near assured. Interestingly, bridge age does not play an overly dominant role in dictating performance of the deck, as some very young bridges have damage and some very old ones do not. It is important to note that Figure 20 does not show the severity of the damage for each bridge. This is due to the difficulty in assigning relative severities among the conditions noted (e.g., abrasion versus exposed rebar versus spalling). A macro trend can be stated, with significant qualification. On average, the larger the negative deviation in cover and older the bridge, the higher the likelihood of damage, which matches intuition. If in subsequent work more data points are added in the -0.4 to -1.1 inches range of cover deviation, and damage is noted in their reports, a more robust conclusion could be drawn on the existence of a damage threshold.

Another interesting area that can be examined using the same data is whether bridges with damage notation had any damage noted in the ten years prior. Answering this question further exposes the influence of bridge age, as well as the rate at which damage accumulates (e.g., linearly, exponentially). An examination of the inspection reports from

10 years prior of the 17 bridges from Figure 20 with deck damage (red triangles) found that only 4 of the 17 (~24%) had previously reported deck damage. Three of the four bridges were the bridges with the most negative cover deviations. These bridges had ages of 48, 34, and 26 years. The remaining bridge was 30 years old. These four bridges had damage first noted at ages of 16, 20, 24, and 38, though they may have been damaged even earlier. That such juvenile bridges experiencing damage gives further support to the notion that bridge age is not the dominant parameter. Sinnce three of the four bridges were the three most negative cover deviations, there may be a high likelihood of damage for bridges with large deviations. It is clear that while performance can be attributed to multiple parameters aside from just the cover deviation, a conservative rejection threshold of a -0.4 inch deviation could be supported by the data.

4.1.1 Corrosion Service Life Modeling Background

The findings from Chapter 3 of this report suggest that a service life model that uses corrosion as the primary degradation mechanism may be most appropriate. Ordinarily, for reasons greatly expanded on in other sources (Pourbaix 2012, Jamali, Angst et al. 2013, Andrade 2019), the steel reinforcement in concrete is protected and corrodes at a negligible rate, a rate frequently described as “passive.” This passivity is the result of the highly alkaline environment inside concrete at the typical steel potentials, which facilitates the formation of a stable oxide layer (Pourbaix 2012), typically called the “passive layer.” Active corrosion, that is to say corrosion that is deemed harmful, is generally the result of either the acidification of the concrete surrounding the steel reinforcement, or the destabilization of the passive layer (in the presence of oxygen) by a sufficient concentration of chloride or other aggressive ions (Andrade 2019). The former is ordinarily caused by the carbonation of the concrete by way of a reaction between calcium hydroxide in the concrete and atmospheric carbon dioxide, termed carbonation-induced corrosion, whereas the latter is often caused by chloride ingress from the environment, termed chloride-induced corrosion. It is important to note that a concrete structure could be undergoing both carbonation and chloride-induced corrosion simultaneously, but generally carbonation-induced corrosion is common in buildings and other structures far from sources of chlorides (including de-icing salts), while chloride-induced corrosion is typically found in bridges and other marine and coastal structures (Angst, Elsener et al. 2012).

As shown in Figure 21, a common framework for corrosion service life modeling of reinforced concrete structures (adaptable to bridge decks) describes the service life (t_{sl}) as consisting of an initiation period (t_i), where there is negligible damage due to the passivity of the steel in the concrete, followed by a propagation period (t_p) where corrosion is ongoing and which ultimately leads to the end of its service life (Tuutti 1982).

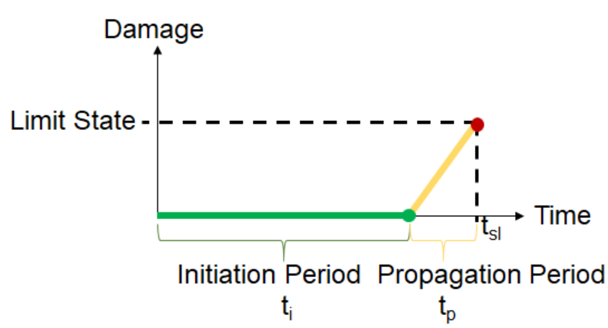


Figure 21. Model. Service life framework.

Different mechanisms and conditions can affect the beginning and end of the initiation and propagation period. These include the rate and amount of chloride ingress, carbonation, and oxygen permeation. The focus of the work in this report will be on chloride-induced corrosion based on the findings from Chapter 3. During the initiation period, chloride ions penetrate the concrete from the external environment, but at the rebar surface, the chloride concentration is below that which is necessary to initiate corrosion (Pillai and Trejo 2005), and thus the steel remains in a passive state. This is known as the chloride theshold (C_t). When the chloride concentration at the rebar surface exceeds C_t, active corrosion initiates, as does the propagation period. The propagation period continues until the deck surface reaches its limit state. The service life of the structure, t_{sl}, is then the summation of the initiation period ti and the propagation period t_p.

It should be noted that the difficulty in reversing the conditions that lead to corrosion in reinforced concrete has resulted in an emphasis on prolonging the time prior to initiation of corrosion, rather than prolonging the subsequent propagation period. The former is typically in the order of many decades, while the latter is normally predicted to be a fraction of a decade. To support this point, the work of Jamali, Angst et al. 2013 can be examined, which showed that when using ten different predictive models, the maximum time to cracking from corrosion initiation was found to be 6 years, under a very modest corrosion rate of 1 μA/cm^2.

In general for any corrosion model, there are four main parameters of interest:

1) Concrete cover (x)
2) Diffusivity (D)
3) Surface Chloride Concentration (C_s)
4) Chloride Threshold (C_t)

The treatment of the concrete cover variability and appropriate default values were addressed in Chapter 2 of this report.

The chloride surface concentration (C_s) is the concentration of chlorides at the exterior surface of the concrete (or as there tends to be a capillary zone, a short distance into the concrete (Schießl, Bamforth et al. 2006)). For bridges exposed to marine conditions (e.g., bridges over salt water), C_s can be estimated with some confidence as the maximum amount of chlorides dissolvable in the concrete pores, which corresponds to approximately 24 kg of chlorides/m^3 of concrete (approximately 40 lb/yd^3). However, a substantial portion of Georgia's bridges are not in aggressive marine environments, but rather service rural roads, urban highways, and small freshwater rivers. For those bridges, the chloride exposure is likely significantly lower. It should be noted however that the use of de-icing salts on roadways can temporarily expose a bridge deck to a maximal C_s despite being otherwise in a low chloride environment, which may aggregate over time. C_s may be determined by measurements from in-service bridges like those performed in studies by Cady and Weyers 1983 and Fanous and Wu 2000, or from coring as per standardized testing such as ASTM C1556-16a (ASTM 2016).

The chloride threshold (C_t) represents the concentration of chlorides at the surface of the rebar at which corrosion initiates. C_t is dependent on many aspects of the concrete and reinforcement, such as the pH of the pore solution, steel potential, and steel composition. Additionally, the use of epoxy coated rebar in Georgia bridge decks may also significantly increase the C_t. This value can be determined from the experimental study developed by Glass and Buenfeld (Glass and Buenfeld 1997) or standardized testing such as ASTM STP1065-EB (Berke, Chaker et al. 1990).

The apparent diffusivity, D, leads to the most variance and uncertainty in the service life modeling. For the case at bar, the apparent diffusivity represents the ease by which the chlorides diffuse through the concrete, with a larger *D* corresponding to poorer quality concrete and less service life. The term apparent refers to the fitting of experimental data to determine *D*. In that manner, apparent diffusivity may indirectly account for phenomenon such as chloride binding. Multiple models exist that aspire to determine *D* from mixture parameters and other factors as expanded upon in (Shafikhani and Chidiac 2019). There is a particular interest in how cracking in reinforced concrete affects *D*, with multiple models having been proposed (Bentz, Garboczi et al. 2013, Sosdean, Marsavina et al. 2016). Even ignoring cracking, the value of *D* for concrete ranges multiple orders of magnitude. It is also known to decrease as the concrete ages due to densification of the concrete matrix and a growing fraction of discontinuous pores.

There exists a variety of standardized tests that can be performed to ascertain the diffusitity, such as: ponding (ASTM C1543-10a) (ASTM 2010), coring/chloride profiles (ASTM C1556-16a) (ASTM 2016), and electrical methods (ASTM C1760-12, ASTM C1202-19, AAHSTO TP95) (ASTM 2012, ASTM 2019, AASHTO 2014). However, this list is not comprehensive, a more in depth investigation of the existing techniques can be found in (Torres-Luque, Bastidas-Arteaga et al. 2014).

4.1.2 Existing Service Life Models

A large variety of reinforced concrete service life programs have been developed (e.g., DuraCrete(Polder and De Rooij 2005), Stadium©, LIFEPRED) as described in detail in (Alexander and Beushausen 2019). There are, however, two service life models that are more commonly used: Life-365™ (Ehlen, Thomas et al. 2009) and National Institute of Standards and Technology (NIST) Chloride-Exposed Steel-Reinforced Concrete Service Life Prediction Program. The above two approaches emerged from a joint workshop in

November 1998 sponsored by NIST, American Concrete Institute (ACI) , and American Society for Testing and Materials (ASTM) called “Models for Predicting Service Life and Life-Cycle Cost of Steel-Reinforced Concrete.” The models were evaluated for suitability for this project. The results of the suitability study are presented in Table 4 and Table 5. From the study, it is apparent that no existing model has all the desired features. Therefore, a hybrid of the different models was created, incorporating features from these models as well as outside sources.

Table 4. Comparison of existing service life model treatment of each parameter.

Parameter	Life-365™	NIST
Concrete Cover, x	Normally Distributed	Normally Distributed
Diffusivity, D	From cores, database, manually inputted, temperature dependent	From mix design, database, user inputted
Chloride Threshold, C_t	Distributed, uses a fraction of weight percentage of cement	Uses a weight percentage of cement
Concrete Surface Chloride Concentration, C_s	Multiple concentrations available based on geographic location of deck	Likely assumes full saturated condition
Time to Corrosion Initiation, T_i	Tends to underestimate	Tends to overestimate
Propagation Time, T_p	Six years	Six years

Table 5. Comparison of the features accounted for in existing service life models.

Feature	Life-365™	NIST
Cracking	No	No
Sealants	Yes	No
Temperature	Yes	No
Chloride Binding	No	Yes
Forecasts After First Corrosion	Yes	No
Alkali-Silica Reaction	No	No
Carbonation	No	No
Freeze Thaw	No	No
Solution Method	Finite Difference	Finite Difference
Concrete Pore Condition	Saturated	Saturated
Pozzolans	Yes	Yes
Validated Against	Fick’s law, bulk data	Bulk empirical data

4.2 One-Dimensional Corrosion Analysis

Simplifying the degradation of reinforced concrete bridges to pure one-dimensional diffusion of chloride ions omits many of the complex interactions in the system, but may still be useful when there exists significant uncertainty of the system parameters (e.g., composition of the pore solution). The accuracy of such a simple approximation has been the subject of investigation (Baroghel-Bouny, Nguyen et al. 2009), where the increased accuracy of the predictions from more sophisticated models is weighed against the corresponding demands for system information. In (Titi and Biondini 2016), it was stated that there is good agreement between the one-dimensional and more complex two-dimensional models when the width of the concrete cross-section is significantly greater than the thickness (in accordance with ratios found in bridge decks).

For systems under pure diffusion in one direction, the concentration at some time and distance can be described by Fick's Second Law in Equation 1.

$$\frac{\partial\varphi}{\partial t} = D\frac{\partial^2\varphi}{\partial x^2} \qquad \text{Eq. 1}$$

In the equation, φ is the concentration at some distance (x) and time (t), and D is a proportionality constant (frequently called diffusivity). For the case at bar, the concentration of interest is that of the chloride ions in the concrete, and the distance of interest is the concrete cover. If the bridge deck is treated as a semi-infinite media, with a constant chloride surface concentration, one-dimensional diffusion, and constant diffusivity, the error function solution is yielded, as shown in Equation 2. In the equation, C is the concentration of chlorides, C_1 is the concentration of chlorides at the surface, C_o is the initial concentration of chlorides initially throughout, x is the depth of interest, and t is time of interest (Crank 1979).

$$\frac{C - C_1}{C_0 - C_1} = erf\frac{x}{2\sqrt{Dt}} \quad \text{Eq. 2}$$

In such an arrangement, the concentration of chlorides in the concrete evolve over time until there exists an equilibrium of concentration throughout the concrete. Of interest is when the concentration of chlorides at rebar depth exceeds the chloride threshold. This point is illustrated in Figure 22, which shows that the threshold was exceeded at the hypothetical 2 inch rebar depth in approximately 6.5 years.

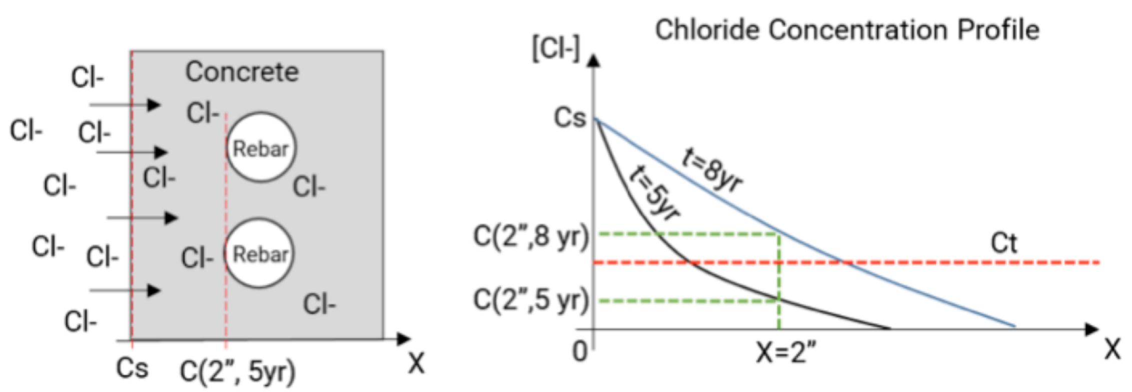


Figure 22. Illustration. Chloride ingress and the resulting chloride profiles for 2 inch rebar depth.

Equation 2 can be simplified further by assuming that $C_0 = 0$ (i.e., no chlorides in the concrete initially), setting C equal to C_t, x equal to the concrete cover, and rearranging the equation to solve for t. With those alterations, the value of t is that which corresponds to the exceedance of the chloride threshold at rebar depth, which should mark the end of the initiation period. This form is used in many practical service life models as in Equation 3 (Bentz and Clifton 1996, Fanous and Wu 2000, Vu and Stewart 2000, Schießl, Bamforth et al. 2006, Titi and Biondini 2016):

$$t_i = \frac{x^2}{4D\left(erf^{-1}\left(1 - \frac{c_t}{c_s}\right)\right)^2} \quad \text{Eq. 3}$$

The service life could therefore be computed as the summation of the result of Eq.3 and an estimate of the propagation period. For the purposes of the work undertaken here, the propagation period was assumed to be 5 years which is within the ordinary range given by (Jamali, Angst et al. 2013).

As noted prior, the main parameters of interest are the concrete cover, the chloride threshold, the surface chloride concentration, and the apparent diffusivity. For the one-dimensional analysis, the most recent GDOT mix design requirements will be used to approximate the parameter values with relationships derived from the literature. The following mix design guidelines were provided by GDOT and can be found, in part, in Section 500 of the Standard Specifications (GDOT 2013) and are provided in Appendix B. A literature review was performed to ascertain reasonable estimates for the other main parameters given in Table 6, Table 7, and Table 8. The values found in the references were all converted to a single set of units, and in the case where the reference provided a range based on percentage of a mix constituent, the value for the Class A and Class D mix were computed and provided as a range.

Table 6. Chloride threshold ranges.

Value	Units	Comments	Reference(s)
1.22-1.3 (0.72-0.77)	lb/yd^3 (kg/m^3)	Estimate based on 0.2% by weight of the cement	(Cady and Weyers 1983, Fanous and Wu 2000, Schießl, Bamforth et al. 2006)
1.97(1.17)	lb/yd^3 (kg/m^3)	Default value from Life-365tm	(Ehlen, Thomas et al. 2009)

Table 7. Surface chloride concentration ranges.

Value	Units	Comments	Reference(s)
21.8 (12.95)	lb/yd^3 (kg/m^3)	Based on 0.1% by weight of concrete, converted from 12950 ppm Cl-. Data supported by compiling the results of coring in 4 states (73 bridges, 688 cores)	(Cady and Weyers 1983, Fanous and Wu 2000, Schießl, Bamforth et al. 2006)
8.31 (4.93)	lb/yd^3 (kg/m^3)	80 Iowa bridge decks	(Ehlen, Thomas et al. 2009)

Table 8. Apparent diffusivity ranges.

Value	Units	Comments	Reference(s)
0.147(3x10^{-12})	in^2/yr (m^2/s)	For w/c=0.45, average ambient temperature of 60F based on work by (Page, Short et al. 1981) on mortar specimens	(Page, Short et al. 1981, Cady and Weyers 1983)
0.489(1x10^{-11})	in^2/yr (m^2/s)	For cement class CEM I 42.5 R with w/c=0.45	(Schießl, Bamforth et al. 2006)
0.240(4.91x10^{-12})	in^2/yr (m^2/s)	Average D measured in a set of Pennsylvania bridges with cover 75 mm (2.95”), w/c<=0.43, minimum cement 400 kg/m3, 15% fly ash by mass of cementitious materials max	(Tikalsky, Pustka et al. 2005)
0.298(6.1x10^{-12})	in^2/yr (m^2/s)	Determined by non-steady state migration tests (NT Build 492) on 100 mm wide, 50 mm thick cores on sound specimens	(Sosdean, Marsavina et al. 2016)
0.050(1.02x10^{-12})	in^2/yr (m^2/s)	Based on analysis from concrete cores taken from 80 Iowa bridge decks	(Fanous and Wu 2000)
0.522-0.614 (1.07-1.26x10^{-11})	in^2/yr (m^2/s)	Based on an equation from the fitting of 10 separate studies using w/cm ratio as the primary input.	(Riding et al. 2013)

w/c = water-cement ratio; NT = NordTest; w/cm = water-cementitious materials ratio

Where there appears to be a spread in the values for each of the parameters, comparisons were made using combinations of the values as shown in Figure 23. The selected values represent 12 permutations with combinations of the C_t values (1.22 and 1.97 lb/yd^3), C_s (8.31 and 21.8 lb/yd^3), and D (min, avg, max: 0.050, 0.301, 0.614 in^2/yr). The permutations were solved according to Equation 3, with the cover incremented by 0.1

inch up to 1 inch from the design cover (2.75 inches) in both directions (1.7-3.7 inch). From the figure, it appears that the envelope for the service life at a design cover 2.75 inches is approximately 7 - 60 years (average of 19 years). From sampling of the decommisioned bridges (see Chapter 3), the average service life from construction to bridge replacement or removal was found to be 60 years (n=286, st. dev=13). Therefore, it appears that the medium and high values for D do not represent field performance well given the selected C_t and C_s values.

Consequently, the analysis will focus on the the four curves that better represent reality with apparent diffusivities of 0.05 in^2/yr. This observation could be due to neglecting more complex phenomena in the model. The width of the envelope appears to be smaller at thinner covers and larger at thicker covers. By averaging the predicted service life at each cover thickness and then calulating the relative difference along the cover range, the average loss in service life per 0.1 inch deviation in cover is 2.4 years (min, max: 1.6, 3.3 yrs).

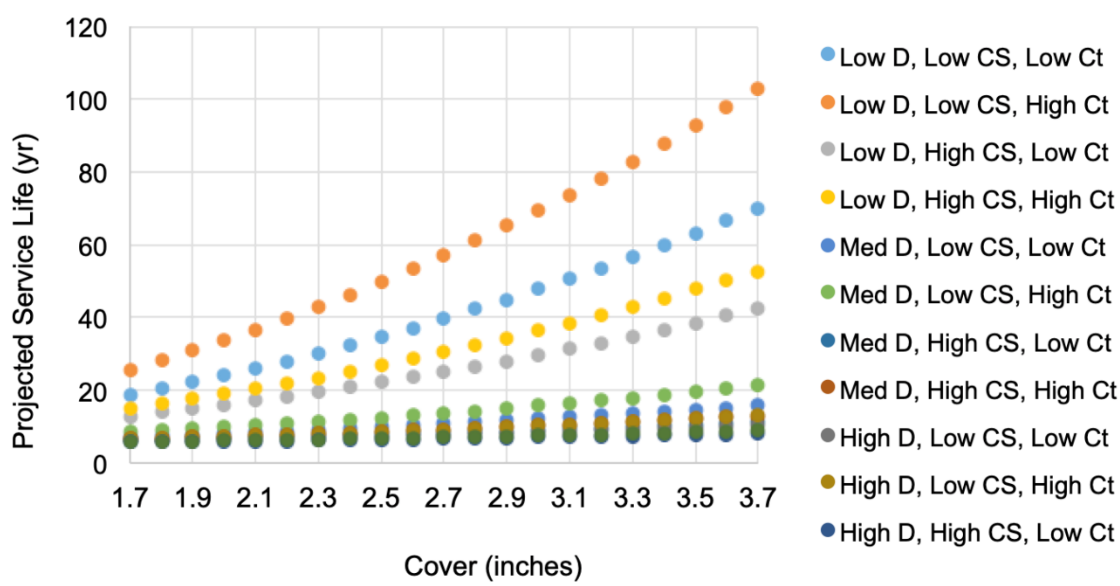


Figure 23. Graph. Results of one-dimensional analysis with varying parameters.

4.2.1 Considerations for Concrete Quality

Using historical data at the GDOT Office of Materials and Testing, mix designs were sampled at random from the 1950s to present, with the belief that the sampled mix designs would be comparable to contemporary designs. In discussions with Mr. Waters, it appears that just prior to 1990, Class A was favored for bridge decks. Since then GDOT has increasingly favored Class D. This change was attributed to a desire for increased strength, as the minimum required compressive strength is 3,000 psi for Class A versus 4,000 psi for Class D. In practical terms, shifting from Class A to Class D results in an increase in the cement factor (and by extension lower w/c), with Class D having approximately 6 percent more cement than Class A per cubic yard of concrete. What is interesting from looking at the Class A records sampled from the 1950s to present was that none of the records indicated that the average measured compressive strength at 28 days was less than 4,000 psi. To support this claim, Table 9 shows the composition and test results for two selected Class A mixes, one from 1976 and one from 1997. The compressive strength for a 2017 Class A mix was also found, and is noted as 4,952 psi.

From our small sampling of 12 Class A mixes, it appears that all would meet the increased strength requirements, though with less margin than Class D mixes. Another interesting general conclusion looking over the sampled records was that less than 20 percent featured any pozzolans (typically Fly Ash). Those mixes that make use of Fly Ash are expected to have longer service lives than those that do not.

Table 9. Concrete mix designs and test results for two sampled Class A mixes.

Parameter	1976 Class A	1997 Class A
Age when tested (days)	28	28
Cement Specific Gravity	3.14	3.14
Fine Aggregate 1 SG	2.63	2.76
Fine Aggregate 2 SG	2.62	-
Course Aggregate 1 SG	2.62	2.75
Cement (lbs/yd^3)	585	611
Sand 1 (lbs/yd^3)	165	1221
Sand 2 (lbs/yd^3)	935	-
Stone 1 (lbs/yd^3)	1875	1921
Water (lbs/yd^3)	295.7	283.9
Design Air %	4	4
Average f'c (lbs/in^2)	5289	4431.5
Average Air %	3.6	4.4
Average Slump (in)	3.75	3.5

SG = specific gravity; f'c = concrete compressive strength

As part of this research, we explored the implication of changing mix classes on the bridge service life for future construction, as that will impact the assessments for contracting. To perform a preliminary analysis of this effect Life-365, an existing commercial service life modeling software, was used. For this experiment, the service life of a single span of an I-85 bridge deck in Gwinnett County, GA was simulated. The one-dimensional slabs and walls model in Life-365 was selected as it was the most appropriate for modeling a reinforced concrete bridge deck. The concrete cover was specified as 2.75 inches, the deck thickness as 8.25 inches, and the span as 70 feet x 40 feet. The base year for the analysis was set as 2017 with a 150 years analysis period. The bridge was set to the climate of Atlanta, GA, with exposure conditions set to a rural highway bridge. Four likely mixes were evaluated, in compliance with the guidance provided by GDOT. These mixes are provided in Appendix B. All other parameters were left at program defaults and are included in Table 10.

Table 10. Mix designs from the exploratory service life modeling.

Case	Minimum	Fly Ash	Stone	Water	Max	f'c
Class A, Cement	611	0	11.2	33	0.490	3000
Class A, with Fly Ash	520	125	11.2	33	0.490	3000
Class D, Cement	650	0	11.2	35	0.445	4000
Class D, with Fly Ash	553	125	11.2	35	0.445	4000

The results of the study are given in Table 11. From the results, the broad conclusion is that a larger increase in service life is expected from the incorporation of fly ash as opposed to the transition between classes. As cement is the most expensive constituent of concrete, it may also be more cost effective to maintain the existing mix class with the addition of fly ash. Future research is needed to further explore and support this conclusion, but the intial results are promising.

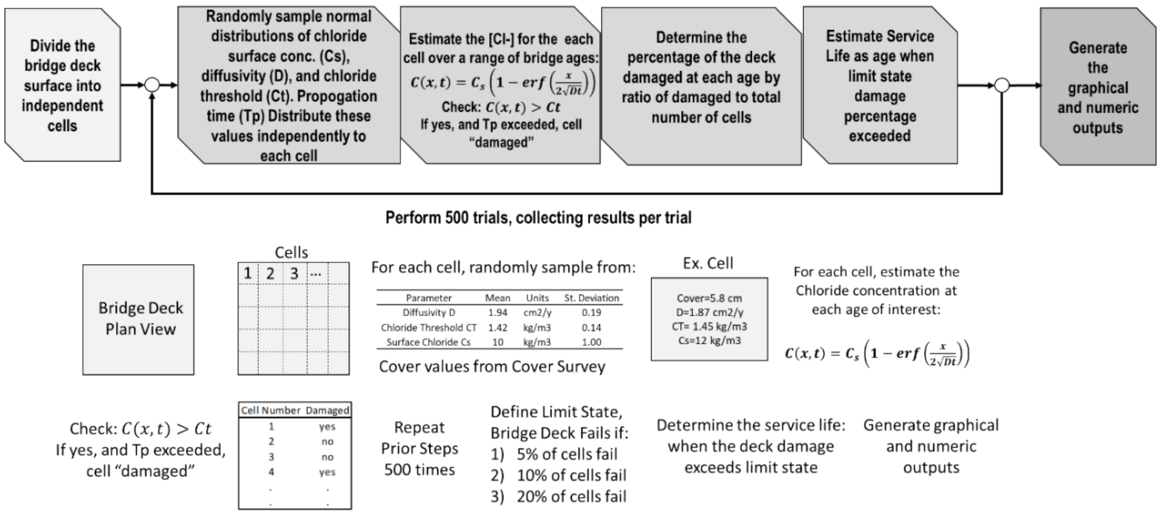
Table 11. Results from the exploratory service life modeling.

Case	Estimated Service Life, Years
Class A, Cement Only	71.2
Class A, with Fly Ash	97.9
Class D, Cement Only	78.2
Class D, with Fly Ash	108.2

4.3 Probabilistic Simulation

While the results from the simple one-dimensional analysis are insightful, a more complex analysis was conducted to confirm and yield a further understanding of the system. Though many models exist that are more sophisticated than that presented in the one-dimensional analysis, the limited information available for the bridge inventory of Georgia limits their application for this project. As will be expanded on in more detail in the recommendations section of this report, there is very limited coordination between the records of interest (e.g., structural plans, cover surveys, mixture proportions), which fall under different jurisdictions within GDOT. It has therefore proven difficult to evaluate the past performance of in-situ bridges as a means to calibrate our models to predict the performance of new construction. Therefore, to address this level of uncertainty in the key parameters, a probabilistic simulation was developed in collaboration with IFSTTAR (The French Institute of Science and Technology for Transport 2008). If in the future more information on the inventory is gathered (i.e., pore solution compositions, resistivity, cracking patterns), other modeling methodologies could be explored.

The methodology developed is presented schematically in Figure 24, and has its basis in the same error function solution of Fick's Second Law presented in Section 4.2. The bridge deck surface is subdivided into smaller surface areas called "cells." Each cell is an equal-sized subdivision of the deck, such that it contains a concrete cover measurement from a cover survey. Therefore, each cell has a maximum size of 10 feet by 10 feet, but may be smaller depending on the concrete cover survey grid, as described in an earlier section of this report.



Parameter	Mean	Units	St. Deviation
Diffusivity D	1.94	cm2/y	0.19
Chloride Threshold CT	1.42	kg/m3	0.14
Surface Chloride Cs	10	kg/m3	1.00

Cell Number	Damaged
1	yes
2	no
3	no
4	yes
.	.
.	.

C_s = chloride surface concentration; D = diffusivity; C_t = chloride threshold; T_p = propogation time; C = chloride concentration; x = concrete cover

Figure 24. Process. Probabilistic modeling methodology.

For each cell, values randomly sampled from normal distributions of the corrosion parameters (surface chloride concentration, chloride threshold, apparent diffusivity, propagation time) are assigned. Next, the performance of the cells are evaluated at the bridge ages of interest (years 6 to 100 in 2 year increment) by way of a modified version of Equation 2. The concentration of chlorides at rebar depth is calculated for each cell and compared to the assigned chloride threshold. If the threshold is exceeded and the propagation period has been satisfied, then the cell is recorded as damaged, if not, it remains in an undamaged state.

The main parameter of interest is the percentage of the deck surface that is predicted to be damaged at a given bridge age. To calculate the percent of the deck damaged, the total number of cells is divided by the number of damaged cells at each bridge age. For each age examined, the entire process described above is repeated 500 times. The results from the 500 iterations are averaged for each bridge age. The final step is to determine the ages at which the limit state criterion are met, namely when the bridge deck has experienced 5 percent, 10 percent, and 15 percent of its area damaged.

The results from the methodology are highly dependent on the model parameters. While most of the key parameters, such as the diffusivity have been discussed previously in the chapter, the determination of an appropriate bridge deck limit state has not been addressed. To ascertain an appropriate limit state, 65 bridges were pseudo-randomly selected from the bridges that had been decommissioned. To qualify for selection, the bridge must have been noted on its final inspection report for at least one of the following defects in its deck: delamination/spalling/patching, abrasion/wear, orcracking. The raw data from this selection is in Appendix C. If attention is paid to solely the categories of delamination, spalls, and patches, and the results are converted to percentage of deck area, then the results presented in Table 12 are achieved.

The results show that the average deck area with delamination/spalls/patches when the bridge is replaced or removed is 10.75% (n=21, standard deviation=22), with a median of 0.72%. If outliers are removed (defined as greater than 2 standard deviations from the mean), the new average becomes 7.1% (n=20, standard deviation=14.8), with a median of 0.66%. It is important to note that it is unclear whether or not the bridges sampled were replaced because of deck deficiencies. For that reason, the decks with higher percentages of damage are more likely the cause of replacement, but for those with very little damage it is likely that there is significant deck service life remaining.

Table 12. Delamination/spall/patch areas in the bridges sampled agnostic of severity

Bridge #	Delamination/Spall/Patched Area (ft^2)
3	2.63
5	83.33
7	0.01
8	0.34
9	50.62
14	1.33
16	38.91
18	0.16
21	0.72
30	0.74
35	0.61
36	6.10
37	0.24
39	0.20
41	1.17
46	0.01
48	0.54
49	0.41
52	35.96
54	1.28
55	0.53

These results can be compared to those found in (Weyers, Fitch et al. 1994), which surveyed the opinion of engineers who make rehabilitation decisions for bridge decks. The authors found that the end of an untreated bridge deck was the point when the level of damage (from spalling, delamination, and asphalt patching) was between 5.8% to 10% of the whole deck, or 9.3% to 13.6% of the worst damaged travel lane. For evaluations undertaken here, the limit states were defined as when 5%, 10%, and 15% of the deck surface was damaged, to capture the above range of values.

Twenty bridges were randomly selected for evaluation using the methodology outlined in Figure 24. The bridges represent a variety of actual cover distributions. The bridges were evaluated against four sets of values for the key parameters, which will be referred to as "cases." The cases are based on the values taken from the literature in Tables 6-8 as well as the performance in Figure 23. The cases were chosen to represent, variations in chloride exposure and variation in steel reinforcement. The value for the diffusivity was kept constant as there is insufficient information to select alternative values. The propagation time was kept at 5 years in alignment with the one-dimensional analysis, and the three main values were assumed to have a normal distribution with a 5% standard deviation. The values used in each case are given in Table 13.

Table 13. Parameters explored in probablistic service life model.

Case	C_s (lbs/yd^3)	C_t (lbs/yd^3)	D (in^2/yr)	St. Deviation
1	8.31	1.97	0.050	5%
2	8.31	3.60	0.050	5%
3	21.8	1.97	0.050	5%
4	21.8	3.60	0.050	5%

The modeling methodology was applied to the twenty selected bridges with the four cases yielding the damage as a function of year, as shown in Figure 25 though Figure 28. The results give an indication as to the predicted percent of the deck that is damaged and can be used to determine at what age bridge limit states are exceeded. The wide range in performance is to be expected as minor variations in the key modeling parameters can have significant effects on the predicted service life, especially over such a large evaluation period. As mentioned prior, the limit states of 5% and 10% of the deck surface were selected to signify the end of the deck's service life. The results given in Figure 25 through Figure 28 are also presented in tabular form in Table 14.

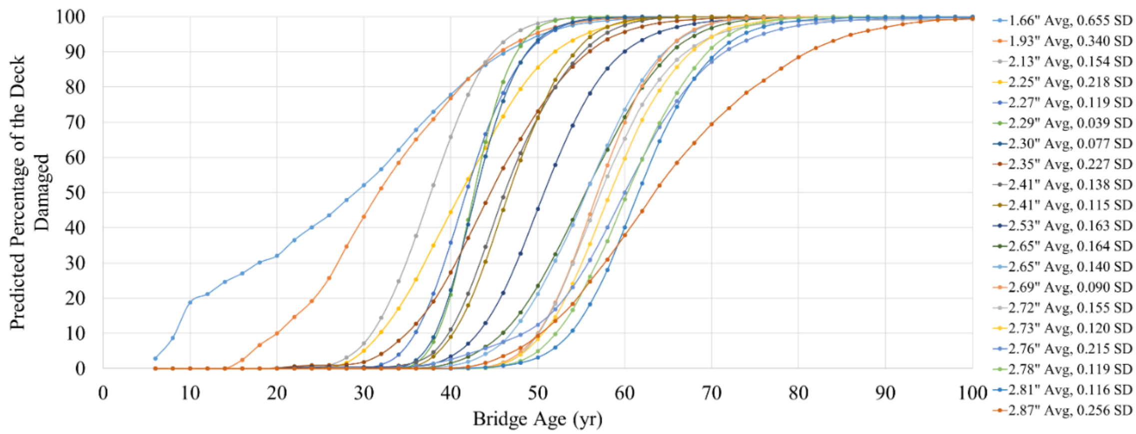


Figure 25. Graph. Case 1 results: predicted percent deck damage over time.

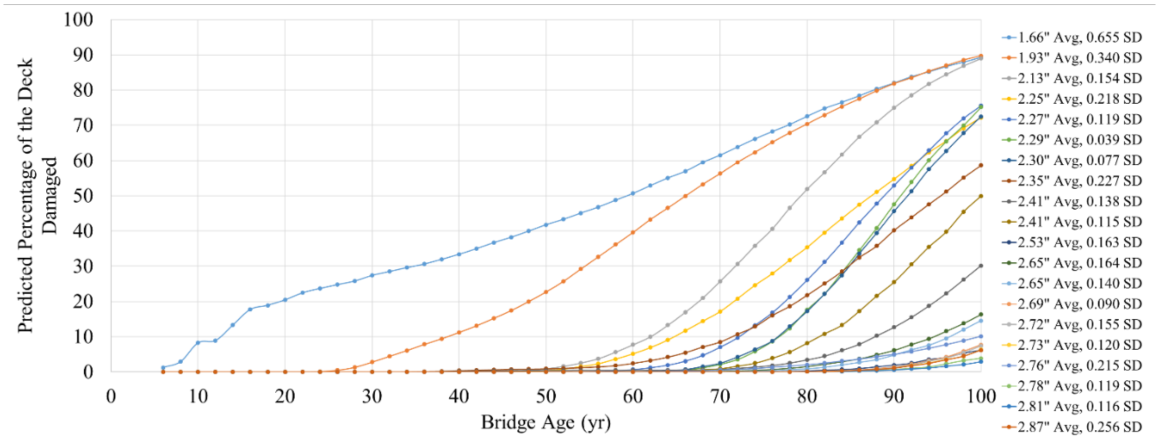


Figure 26. Graph. Case 2 results: predicted percent deck damage over time.

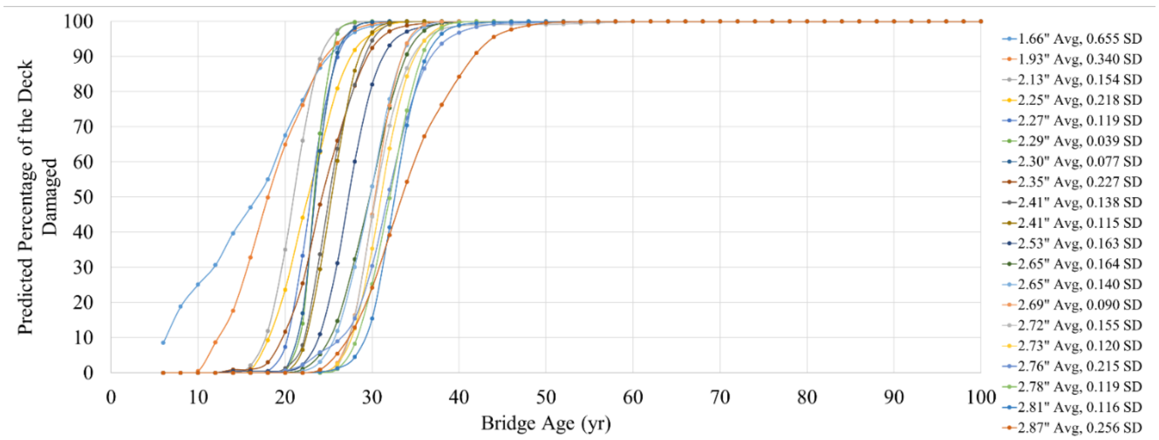


Figure 27. Graph. Case 3 results: predicted percent deck damage over time.

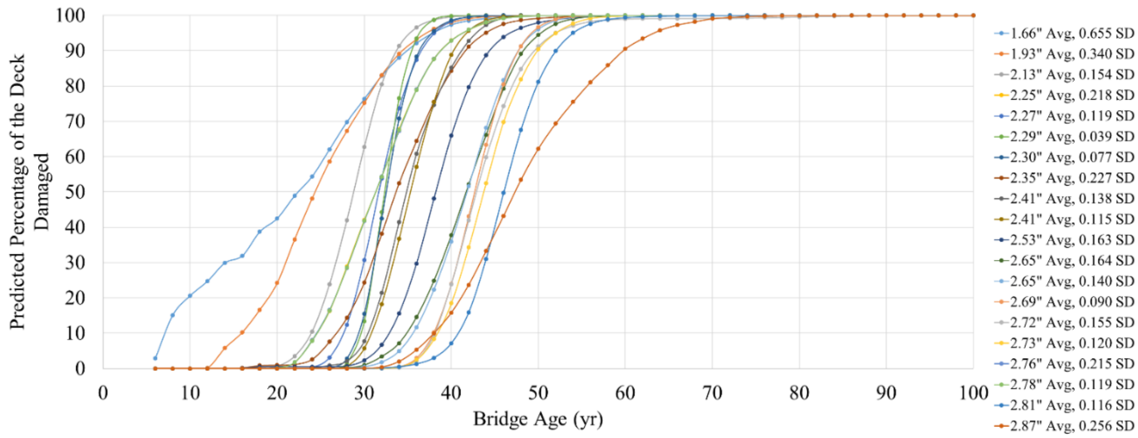


Figure 28. Graph. Case 4 results: predicted percent deck damage over time.

Table 14. Predicted service lives in years for the bridges evaluated based on the case and selected limit state.

Bridge	Cover Average (in)	Cover Standard Deviation (in)	Service Life, Years - 5% Damage Limit State				Service Life, Years - 10% Damage Limit State			
			Case 1	Case 2	Case 3	Case 4	Case 1	Case 2	Case 3	Case 4
1	1.66	0.65	7	9	6	6	8	12	6	7
2	1.93	0.34	17	33	11	14	20	39	12	16
3	2.13	0.15	29	57	17	22	31	62	18	24
4	2.25	0.22	30	60	17	23	32	65	18	24
5	2.27	0.12	34	68	19	26	36	72	20	27
6	2.29	0.04	37	73	21	29	38	77	21	29
7	2.30	0.08	37	73	21	28	38	77	21	29
8	2.35	0.23	32	65	18	25	35	71	20	27
9	2.41	0.14	38	83	21	29	40	88	22	30
10	2.41	0.12	39	77	22	30	40	81	22	31
11	2.53	0.16	41	97	23	31	43	100*	24	33
12	2.65	0.16	43	88	24	33	46	95	25	35
13	2.65	0.14	45	90	24	34	47	96	26	36
14	2.69	0.09	48	97	26	37	50	100*	27	38
15	2.72	0.15	48	97	26	37	50	100*	27	38
16	2.73	0.12	48	99	26	37	51	100*	27	38
17	2.76	0.22	43	90	24	23	48	100	26	24
18	2.78	0.12	50	100*	27	23	52	100*	28	25
19	2.81	0.12	51	100*	28	39	54	100*	29	41
20	2.87	0.26	47	100*	26	36	50	100*	27	38

* Projected service lives in excess of the evaluation period of 100 years.

The results for the four cases were averaged for each limit state and then plotted according to the average cover for each bridge, as shown in Figure 29. From the figure, the predicted service life can be seen for a given average cover. Note the slight variation in the fitting between the 5% and 10% limit states with otherwise good agreement. The predicted service life for 2.75 inches is in the order of the 60 year average service life observed in the early portions of this report.

To illustrate the use of the fitted equations from Figure 29, a test case is presented. Consider a bridge with the design cover of 2.75 inches. According to the 10% damage limit state, the expected service life is approximately 53 years. To evaluate the effects of variation in the average cover, the fitted equation can be used to generate results as shown in Table 15.

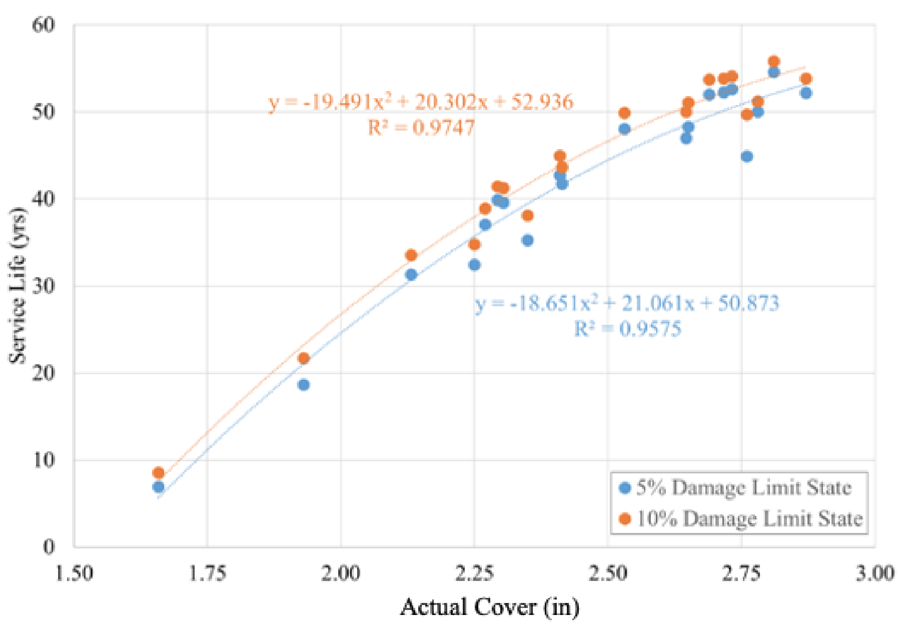


Figure 29. Graph. Results from the one-dimensional analysis, plotting the predicted service life based on the sampled cover surveys and the 5% and 10% limit states. The data is fitted with polynomial trendlines, with the equations displayed on the figure.

Table 15. Results from Figure 29, normalized to 2.75 inches design cover.

Cover Deviation from 2.75 inch (in)	Predicted Service Life 10 Percent Damage Limit (Years)	Change in Predicted Service Life from that of Design Cover (Years)
0.5	58	5
0.4	58	5
0.3	57	4
0.2	56	3
0.1	55	2
0	53	0
-0.1	51	-2
-0.2	48	-5
-0.3	45	-8
-0.4	42	-11
-0.5	38	-15

Table 15 shows that having an average cover that is 0.1 inch greater or less than the design cover (2.75 inches) results in a two year change to the predicted service life. In fact the average change in service life is approximately two years per 0.1 inch, in agreement with the findings in the one-dimensional analysis. It appears that the service life gains of additional cover diminish more readily than the service life losses expected from thinner cover. The results from these modeling approaches serve as a foundation for the contractual remedies explored in the next chapter.

5. CONTRACTUAL RECOMMENDATIONS

This chapter discusses the appropriateness of different contracting mechanisms that may be used in future construction to achieve better cover control. The chapter relies heavily on the findings of the previous three chapters to provide a rationale and data for the approaches. Section 5.1 presents the legal landscape related to public works contracts that were investigated in terms of general contract law, the specific statutes in Georgia as well as other states, and the relevant Georgia case law. Section 5.2 evaluates contracting methods that are applicable for proper cover control and provides a discussion on implementing the contracting methods into the existing GDOT provisions. Finally, Section 5.3 provides an example of the most applicable method(s).

5.1 Applicable Contract Law

5.1.1 Remedies

As a starting point, it is important to discuss the general remedies available for breach of contract. The Georgia Code Title 13 Chapter 6, Damages and Cost generally states that permissible damages broadly fit into five categories which are covered in sections § 13-6-6 through § 13-6-10 (Georgia Code 2018) and summarized in Table 16.

Table 16. Georgia code damages and remedies.

Damage Type	Relevant Section	Language
Nominal	GA Code § 13-6-6	In every case of breach of contract the injured party has a right to damages, but if there has been no actual damage, the injured party may recover nominal damages sufficient to cover the costs of bringing the action.
Liquidated	GA Code § 13-6-7	If the parties agree in their contract what the damages for a breach shall be, they are said to be liquidated and, unless the agreement violates some principle of law, the parties are bound thereby.
Remote or Consequential	GA Code § 13-6-8	Remote or consequential damages are not recoverable unless they can be traced solely to the breach of the contract or unless they are capable of exact computation, such as the profits, which are the immediate fruit of the contract, and are independent of any collateral enterprise entered into in contemplation of the contract.
Expenses Necessary for Compliance	GA Code § 13-6-9	Any necessary expense, which one of two contracting parties incurs in complying with the contract may be recovered as damages.
Exemplary	GA Code § 13-6-10	Unless otherwise provided by law, exemplary damages shall never be allowed in cases arising on contracts.

It is likely that constructing a bridge with improper cover control would constitute a breach of contract by the contractor and thus expose the contractor to legal liability. A breach of contract by improper cover control would not be a nominal damage as the state incurred actual damage (i.e., diminished asset). An improper cover control breach could be eligible for remote or consequential damage, though it would be difficult to ascertain that the negative effects suffered were solely because of poor cover and lack of an appropriate remedy. Expenses necessary for compliance may apply but only in narrow circumstances, such as if the DOT hires another firm to correct the improper work and seeks to recover the costs against the original contractor. Exemplary damages are prohibited. Of the damages listed, the most applicable to a breach for improper cover control is liquidated damages; this is due to the difficulty in quantifying with certainty the costs incurred by a breach in advance.

5.1.2 Public Works Contracts - Georgia

Based on the analysis above, it is unsurprising that the Georgia Code only mentions liquidated damages in relations to public works projects. The most relevant statutes can be found in §13-10-70 of the Georgia Code, "Liquidate damages for late completion and incentives for early completion (Georgia Code 2018)," which offers the following guidance:

> *"Public works construction contracts may include both liquidated damages provisions for late construction project completion and incentive provisions for early construction project completion when the project schedule is deemed to have value. The terms of the liquidated damages provisions and the incentive provisions shall be established in advance as a part of the construction contract and included within the terms of the bid or proposal."*

Section 13-10-70 emphasizes that liquidated damages must be agreed upon in advance, and also provides for both an incentive for early construction as well as a disincentive for late completion. It is important to have a corresponding incentive for every provision with a disincentive as the courts have ruled that provisions that function solely as a penalty are prohibited (as will be expanded on in the next section). Section 13-10-70 is a clear example of an incentive/disincentive (I/D) contract mechanism, intended to reduce the construction time for public works projects. However, the prescribed mechanism would be inappropriate for our purposes, namely to incentivize construction quality, which is not related to reducing construction time. It is therefore useful to examine the statutory language used in other states, which may present provisions that could be readily adapted to the goals of this project.

5.1.3 Public Works Contracts – Other States

The state codes and statutes of Virginia, Texas, Indiana, Florida, Ohio, Utah, and California were examined. In general, it appears that the states defer to the Universal Commercial Code (UCC), in whole or with modification, to serve as the general basis for their contracting laws, with specific amendments by statute. The overall consensus is that there may be no penalty clauses in contracts without the prospect of receiving a bonus. Liquidated damages are generally permissible so long as they are not solely used as a penalty.

Virginia, Texas, and Indiana provide no specific statutory requirements for public works contracts. In examining Florida statutes, the most relevant section is FL Stat § 337.18 (Florida Statutes 2019), which allows for liquidated damages, and in the case where time is of the essence, an incentive payment is permissible. Ohio § 5525.20 provides the following incentive and disincentive provisions for critical construction projects (Ohio Revised Code 2019):

> *"…the director of transportation may include incentive and disincentive provisions in contracts the director executes for projects or portions or phases of projects that involve any of the following:*
>
> > *(1) A major bridge out of service;*
> >
> > *(2) A lengthy detour;*
> >
> > *(3) Excessive disruption to traffic;*

(4) A significant impact on public safety;

(5) A link that completes a segment of a highway.

As used in this section, 'incentive and disincentive provisions' means provisions under which the contractor would be compensated a certain amount of money for each day specified critical work is completed ahead of schedule or under which the contractor would be assessed a deduction for each day the specified critical work is completed behind schedule. The director also may elect to compensate the contractor in the form of a lump sum incentive for completing critical work ahead of schedule."

Utah gives wide latitude to the remedies which are permissible in section 63G-6a-1210 (Utah Code 2018):

"Contract provisions for incentives, damages, and penalties.

A procurement unit may include in a contract terms that provide for:

(1) incentives, including bonuses;

(2) payment of damages, including liquidated damages; or

(3) penalties."

California appears to have very explicit language in terms of incentivizing early construction and reducing costs or inconvenience to the public. Also, the code is unique in that it specifically addresses alternative delivery methods such as Design-Bid-Build. Two relevant examples are those given in CA Pub Count Code § 7101 and CA Civ Code § 1671:

"The state or any other public entity in any public works contract awarded to the lowest bidder, may provide for the payment of extra compensation to the contractor for the cost reduction changes in the plans and specifications for the project made pursuant to a proposal submitted by the contractor. The extra compensation to the contractor shall be

50 percent of the net savings in construction costs as determined by the public entity. For projects under the supervision of the Department of Transportation or local or regional transportation entities, the extra compensation to the contractor shall be 60 percent of the net savings, if the cost reduction changes significantly reduce or avoid traffic congestion during construction of the project, in the opinion of the public entity. The contractor may not be required to perform the changes contained in an eligible change proposal submitted in compliance with the provisions of the contract unless the proposal was accepted by the public entity (California Code 2018).”

“…(b) Except as provided in subdivision (c), a provision in a contract liquidating the damages for the breach of the contract is valid unless the party seeking to invalidate the provision establishes that the provision was unreasonable under the circumstances existing at the time the contract was made. (California Code 2018)”

In summary, there appears to be no significant statutory barriers to adopting new contacting methods in Georgia. Although the contract mechanism may need to be based on the permissible liquidated damages statute, it appears that most states have similar language related to the permission of liquidated damages for public works contracts. None of the states provide a provision that could be immediately applicable to improving cover control in bridge decks due to their use of an incentive that is proportional to construction time.

5.1.4 Relevant Georgia Case Law

In order to investigate the common law landscape for public works contracts, an investigation of relevant case law was undertaken for the State of Georgia. Two relevant cases were found to be applicable to this effort.

In the first case, Southeastern Land Fund v. Real Estate World (237 Ga. 227 1976) considered whether a provision in a real estate sales contract constituted an enforceable liquidated damage provision or a penalty. The provision in question stipulated that $5,000 paid in earnest money to the seller was partial liquidated damages in the case of default, as a means to collect the proceeds of the indebtedness owed. When the buyer defaulted, the seller sued for more damages than the 5,000 dollars, claiming that they were entitled to pursue any and all legal remedies including, but not limited to, the $5,000. The case reaffirmed the requirements for a liquidated damages provision. "First, the injury caused by the breach must be difficult or impossible of accurate estimation; second, the parties must intend to provide for damages rather than for a penalty; and third, the sum stipulated must be a reasonable pre-estimate of the probable loss (Calamari and Perillo 1998)." This case addressed the intent of the parties for the second requirement, wherein the court found that in this particular case the seller intended to retain the right to other damages rather than liquidated damages, and so the provision was unenforceable. The court made the point that liquidated damages can be enforced in addition to other remedies given in explicit language in the contract, otherwise the provision may instead be a penalty, and thus unenforceable.

The second case was Fortune Bridge Co. v. Department of Transportation (242 Ga. 531 1978). Fortune Bridge Company was awarded a $1M contract to build three bridges and a roadway for U.S. 19 in Georgia within a period of 620 days. The bridges were eventually constructed with a delay of about a year, so GDOT withheld $73,000 ($200/day). The case appeared in front of the Supreme Court of Georgia, where the liquidated damages provision was upheld due to the inability to calculate the actual and consequential damages of a breach. This case supports the notion that a liquidated damages clause may be enforceable for cover deficiencies if GDOT was unable to accurately determine the actual damages in advance.

5.2 Contracting Methods and Implementation

State transportation agencies have employed contracting methods to achieve construction goals, such as reduced construction time, reduced project cost, or quality assurance. Some of the main contracting methods used to achieve those goals are given in Table 17. The strengths and weaknesses of the methods have tailored their application to public works projects. An in-depth discussion of the provisions and how they relate to this concrete cover is given in the next subsections.

Table 17. Contracting methods used by state agencies.

Method	Characteristics	Typical Uses
Incentives/Disincentives	Calculate a per day cost for early project delivery or delay related to the direct and indirect cost of project.	To achieve faster project delivery. Often in urban projects with high cost for delays. Used in highway construction and refurbishing.
Warranties	Requires contractor to repair or replace work if it fails to meet expected service life.	In cases where there is an interest in ensuring quality, examples include warranties on asphalt pavement. Generally short-term projects.
Design-Build-(Finance)-Operate-Maintain Frameworks	Contract features a requirement that the contractor operates and maintains the asset after construction. Shifts risk to the contractor and incentivizes quality construction.	In cases where it is feasible to shift operation and maintenance of the infrastructure to the contractor.
Acceptance/Adjustable Plans	Contract stipulates a testing regime that the work is subject to. The results of the testing can lead to a pass/fail judgment for acceptance plans, or a reduction/increase in payment due for the adjustable plans.	Used in pavement construction or other cases where quality assurance is the primary goal.

5.2.1 Incentive/Disincentives (I/D)

Incentive/Disincentive provisions are generally intended to reduce construction time and, in some cases, cost. The liquidated damages provisions mentioned in many state codes are the embodiment of the I/D provision, representing the compensation or cost incurred for changes in the delivery date of the project. In the case of the California Code, another form of I/D provision is given. In this case, it is solely an incentive provision, whereby the cost savings are split between the state and the contractor. For our particular application, namely cover control, I/D provisions such as the examples given are not well suited to ensuring cover control as we intend to improve construction quality instead of construction speed or cost.

5.2.2 Warranties

As noted by (Thompson, Anderson et al. 2002), multiple states use warranties for a variety of public works projects, ranging from roadway quality assurance to steel bridge painting quality. The advantage of warranties is that it allows the contractor to optimize the construction process, which may result in more innovation and reduced cost. Among the applications for warranties in the public works environment, the length of the warranty period (an important parameter to optimize) is noted to be between 2 and 20 years in the various applications. This period of time would be inadequate for the case of cover control as the effects are evident after longer periods of time and only toward the end of the service life. It would be unreasonable to expect a contractor to warranty a bridge for such a long period of time since the contractor may no longer be in business. For this reason, warranties do not provide a good option for the application of this study.

5.2.3 Design-Build-(Finance)-Operate-Maintain (DBOM) Frameworks

In recent years, the State of Georgia has passed a law that allows the private finance and operation of infrastructure (including bridges) in the State (Georgia Code 2013). The legislation sets requirements that GDOT annually identify projects that "afford the greatest gains in congestion mitigation or promotion of economic development" that would be appropriate for a public-private partnership (P3). The goals of the P3 initiative is to seek "innovative project delivery and innovative financing solutions from the private sector to meet the State's transportation needs." DBOM would represent a P3 arrangement. For this application, the public agency (or private industry) finances the construction of the bridge, with a separate entity design-build-operate-maintaining the structure for a period of time. While there are no known P3 projects that are currently being constructed that may change in the future.

To give a sense of how a P3 bridge could be realized, the Confederation Bridge in Canada can be taken as a case study (Cheung, Tadros et al. 1997). The Confederation Bridge was completed in 1997, having been entirely funded through a private consortium. In return for constructing the bridge, the consortium receives tolls on the bridge as well as an annual payment ($44M for 33 years) from the Canadian government. In 2032, the bridge will revert to federal government ownership, but in the intervening time the consortium is responsible for the operation and maintenance of the structure.

If GDOT were to transition to a DBOM framework for future construction, the impacts of insufficient cover may be borne by the contractor, which will have an incentive to build a quality bridge to avoid that liability. There are, however, general policy and legal concerns that will need to be addressed before DBOM frameworks become widely adopted for bridge construction.

5.2.4 Acceptance/Adjustable Plans

An acceptance plan is one that stipulates a testing regime for the parameter of interest or product, and then based on the results and an acceptance threshold, accepts or rejects the product. An adjustable payment plan uses the same methodology except instead of the binary acceptance or rejection decision, the payment for the product is adjusted based on the results. Both of these methodologies have been used by multiple DOTs for ensuring asphalt roadway quality and painting quality of steel bridges (Tuggle 1992). These methodologies could be readily adapted for cover control. In fact, arguably the status quo is an acceptance/adjustable payment plan. If the cover control is inadequate and cannot be remediated, the engineer is empowered to reject the work similar to an acceptance plan as in Section 105.12 of the Standard Specifications (GDOT 2013). The engineer may also choose to reduce payment for the deck in a manner similar to an adjustable payment plan as in Section 105.12 of the Standard Specifications (GDOT 2013).

For both acceptance and adjustable payment plans the testing methodology is paramount. In the case at bar, the testing methodology would involve sampling the plastic deck surface to determine the cover distribution. The required number of samples and how the locations are chosen (i.e., randomly) would need to be stipulated. More samples would yield greater certainty that unacceptable work is not being unintentionally accepted or that acceptable quality work is mistakenly rejected. For an adjustable payment plan, more sampling would reduce the likelihood of underpaying the contractor for cover control that is at the acceptance limit. It could also be combined with a bonus in the payment scheme for work over the acceptable limit that offsets the risk to the contractor.

For inspiration on how the testing methodologies could look for both of these contracting methods, GDT 73 Method C (random selection of roadway concrete samples) (GDOT 2018) or SOP 46 (procedure for calculating pay reduction for failing roadway and bridge approach smoothness) (GDOT 2018) are good examples.

The methodology from GDT 73 Method C could be readily adapted for cover control compliance testing with the simple substitution of a span of the deck as the lot boundary and measuring the cover thickness instead of measuring the thickness of the roadway. The method provides tables for randomly selected the locations for the depth checks within a subsection of the work termed a “sub lot.” The method provides for a revision to the payment for the roadway if cores measure more than 0.2 inch deficient to the construction plans, in Subsection 430.5.01.A (cover pavement thickness deficiency).

In SOP 46, the pay reduction for substandard road smoothness is computed by subtracting the ratio of the specified roadway smoothness to the actual road smoothness for each failing mile section from one. For our purposes, the ratio of the actual average cover to the design cover could be subtracted from the full pay value. The pay factor reduction is then used to de-rate the payment for all square yards of product in the failing mile section(s).

For ensuring cover control the most obvious sampling scheme is the one currently being used, namely testing between girder bays and along the surface in approximately 10-foot intervals. The problem with this scheme is that it is not random sampling, but rather systematic, particularly if the sampling intervals are accurately reproduced along the deck.

That is not to say that the data collected would be worthless, but the testing scheme should incorporate random checks in addition to the existing measurements. Future work should be undertaken to relate the number of testing locations to the accuracy of acceptance/adjustable payment plans.

5.3 Adjustable Payment Plan Draft Specification

This section demonstrates the process for creating an adjustable payment plan and the considerations for the draft standard operating procedure (SOP) provided in Appendix D. The process for creating an acceptance plan is found in (AASHTO 2018). For stipulating an adjustable payment plan, the information from the previous sections becomes vital. The process that will be followed is outlined in AAHSTO Specification R9-05 (AASHTO 2018). The specification requires the designation of a few key parameters. The first parameter that must be defined is the acceptable quality limit (AQL). It is recommended in a process with a stipulated target value (e.g., cover), to set the AQL equal to the target value. For this example AQL is set to the design cover of 2.75 inches.

The next parameter that must be defined is the percent within limits (PWL), which refers to the percentage of the true cover distribution of the lot that is within specifications. The specification recommends that 90 PWL be used. That is to say, that if the sample were to have the target value as the mean and the designated standard deviation for the process, 90% of the true cover distibution, not necessarily the sampled distribution would be within the specified limits.

It is therefore important to determine the appropriate standard deviation and lot size. The lot size is selected to be a bridge deck span. For determining the appropriate standard deviation to use for cover control, first the process standard deviation must be determined. The standard deviations from the 36 spans used in earlier portions of this report were averaged and resulted in a value of 0.0414 inch. The average target miss by the contractor was -0.07 inch, which is not significant enough to warrant consideration. That is to say, that contractors have proven capable of accurately meeting the design cover. Next, the standard deviation of the center needs to be computed, which is the standard deviation of the deviations from the design cover. For the above 36 spans, the standard deviation for the center was determined to be 0.143 inch. The appropriate standard deviation, termed the combined standard deviation in the specification, is then found by Equation 4. For this example, the combined standard deviation was found to be 0.15 inch. Therefore if the mean is set as the design cover and the standard deviation is 0.15 inch, for a PWL of 90, the lower specification limit (LSL) will be 2.51 inches and the upper specification limit (USL) will be 3.00 inches.

$$\sigma_{combined} = \sqrt{{\sigma_{center}}^2 + {\sigma_{process}}^2} \qquad \text{Eq. 4}$$

Next, the rejection quality level (RQL) needs to be specified. The RQL represents the percentage of samples that would be within specification limits if the mean of the sampling was the rejection threshold and the standard deviation was 0.15 inch. The specification states that RQLs are generally between 70 and 30 PWL. The GDOT Standard Specifications allow for an up to 0.25 inch deviation from the plans in cover pre-grinding and 0.5 inch deviation post-grinding. It has been noted previously that the cover measurements are taken pre-grinding when confirming concrete cover, so the 0.25 inch will be considered the RQL threshold. Therefore, the percent of the distribution if the mean were 2.50 inches as well as 3.00 inches needs to be determined. Figure 30 below represents these limits graphically. The area of interest is the area of each outside distributions falling within the LSL and USL.

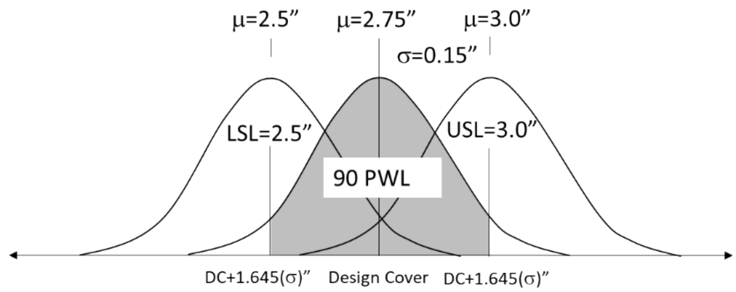


Figure 30. Illustration. Specification limits for cover.

To calculate the area, a Z table is employed using the Z value for the LSL and USL in relation to the two exterior distributions. The area of each exterior distribution within the LSL and USL is 50%, therefore the RQL is 50. The specification states that RQLs are generally between 70 and 30 PWL, so the chosen RQL is within the normal range.

Next, two important parameters need to be established: the number of random samples per lot, *n*, and the form of the pay equation. When selecting *n*, the primary concern is the risk that the sampling of the sublot fails to approximate the true cover distribution, and by extension, the payment for the work will not reflect the delivered quality. As the number of samples increases, the corresponding risk of having an unrepresentative sample decreases, but concurrently the time and expense for the sampling increases. In practice, a balance between the two interests is struck, with a normal random sampling range in highway construction and materials acceptance plans being between three and seven samples (Gharaibeh et. al 2010).

ASTM E112-17 prescribes a means of calculating the sample size needed to estimate within a desired precision (ASTM 2017). The relevant formulation is given in Eq.

5. It is important to note that the equation assumes the true distribution is normal, or approximately normal, which was demonstrated in previous sections of this report.

$$n = \left(\frac{3\sigma_o}{E}\right)^2 \qquad \text{Eq. 5}$$

In Equation 5, n is the sample size, σ_o is an advance estimate of the standard deviation, and E is the maximum acceptable deviation between the true average and the sample average. If σ_o is assumed to be the 0.15 inch as determined through the process outlined previously in this section, and E is selected as 0.1 inch, then n is approximately 20. It is important to note that E is the maximum acceptable deviation, and that even if 0.1 inch is selected, the observed deviations will generally be some value smaller. Interestingly, if the value for E is 0.2 inch instead, then n drops to approximately 5, which is within the ordinary range of 3 to 7. For the purposes of this report, considering the narrow range of +/-1/4 inch average deviation for the vast majority of decks, n will be set at 20 to conserve the granularity of the testing, with the understanding that GDOT may in the future consider less sampling to conserve cost and resources.

The next consideration is the manner in which the 20 random samples are selected. For this topic, the reader is refered to GDT 73 Method C “Random Selection of Roadway Concrete Samples”, which outlines a procedure by which random samples may be selected, with the modification that the lot boundaries are the spans and that each lot is evaluated independently (GDOT 2018). There are many alternative ways to randomly sample the 20 points on each span.

The next matter is selecting the form of the pay equation. One of the first concerns is whether or not there should be a “bonus” incorporated in the pay equation in addition to penalties. In other words, should both an incentive and disincentive be included. Burati et al provide a strong argument for the need of a positive incentive as well as a penalty. Namely that the positive incentive is required to ensure that on average 100% is paid for the acceptable quality limit (AQL) work and thus payment is not unfairly biased downward. This is done to promote adoption by the construction industry and the added economic value from the improved quality assurance/quality control (QA/QC) (Burati 2003). Novak et al note in their work developing a performance-related specification for controlling the compressive strength of deck concrete for the Vermont Agency of Transportation (VTrans) that a net overpayment of 3% was budgeted and implemented in their system (Novak 2018). This net overpayment was intended to incentivize industry partners to improve their technology and practices and reduce the likelihood of the industry simply raising bid prices to offset expected losses from delivering consitent with their historical norms (Novak 2018). While the case has been made for a reward as well as a penalty, some of the equations evaluated will be devoid of the bonus provision, for purposes of insight as well as for implementation in cases where net overpayments have not been budgeted.

The first pay factor equation is evaluated as shown in Equation 6, which is the default provided in the specification (AAHSTO 2018, Burati 2003). The equation assumes a linear form, with an apparent bonus that cannot exceed 5%. The expected payment just above the RQL is 80%, which represents the floor for the pay factor (PF).

$$PF(\%) = 55 + 0.50(PWL) \qquad \text{Eq. 6}$$

The second pay factor equation evaluated is a simple modification of Eq. 6, where the intercept is reduced from 55 to 50, thereby removing any possibility of a bonus as shown in Equation 7.

$$PF(\%) = 50 + 0.50(PWL) \qquad \text{Eq. 7}$$

The third and fourth pay factor equations are based on the comparable SOP 46, "Procedure for Calculating Pay Reduction for Failing Roadway and Bridge Approach Smoothness" (GDOT 2018). There are some modifications to the SOP 46, namely average cover and design cover replace correction smoothness and actual smoothness, and the intended result when the ratio of the average to design cover exceeds one. It is on this point that the equation is split into two, with the third pay factor equation (Equation 8) having no special provision. For the fourth equation (Equation 9), a stepwise formulation is adopted to prevent overpayment when the ratio is greater than 1.

$$PF(\%) = \left(\frac{AC}{DC}\right) * 100 \qquad \text{Eq. 8}$$

$$PF(\%) = \begin{cases} \left(\frac{AC}{DC}\right) * 100, & \left(\frac{AC}{DC}\right) \leq 1 \\ 100, & \left(\frac{AC}{DC}\right) > 1 \end{cases} \qquad \text{Eq. 9}$$

In the equations, AC is the average cover and DC is the design cover. It should be apparent that this formulation lacks many of the benefits provided by equations based on percent within limits (PWL), which incorporate the variability of the cover, and seek to better estimate the true impact of the cover distribution.

5.4 Evaluating the Pay Factor Equations

There are many conceivable ways that the proposed pay factors could be evaluated, though they all should rely on the stated objectives of the plan and the feedback of the stakeholders. For this study, the input of the stakeholders have not been thouroughly investigated, which is a task for future work, but rather comparable payment plans that have been implemented in other states were examined to gain insight. Novak et al state that Vermont Agency of Transportation (VTrans) decided that there should be a broad and conservative peak (centered on the target value) in its pay structure, with a linear transition as the observed distribution deviates from the target (Novak 2018). A very similar broad payment structure (a conservative peak with linear transitions) can also be seen in the work of Buddhavarapu et al and their refinement of an existing adjustable payment plan for hot mix asphalt for the Texas Department of Trasnportation (TxDOT) (Buddhavarapu 2014). Therefore, the pay factor equations should be mild and gradual in TxDOT pay adjustments and not overly sensitive or harsh.

With the above in mind, a series of 40 bridge spans, from bridges constructed in 1978 to 2019 were randomly selected to serve as case studies by which to evaluate the pay factor equations. It is regretable that the cover surveys are not randomly selected data points, but rather are taken in a systematic manner as described prior, and that the dimensions of the spans varry considerably (making economic comparisons more challenging). These defects will be ameliorated in actual practice, but for the purposes of evaluating the pay factor equations it can be assumed that the current systematic cover sampling sufficiently captures the true cover distribution, which is the intent. One final point is that the severity of a pay reduction may be largely dependent on the cost of the bridge deck. To incorporate this effect in a simple manner, it will be assumed that a square foot of deck has the same cost in each bridge and that the number of data points per cover survey is a reasonable estimator of the size, and thus cost of the bridge. To put some reasonable numbers to the cost, the Florida Department of Transportation (FDOT) provides some values that can be used to calculate costs (FDOT 2014). From FDOT, for a medium and long simple span bridge with a concrete deck and steel girders, the cost per square foot for new construction is between \$125- \$142 (midpoint of \$133). For a similar bridge with pre-stressed girders instead of steel, the cost per square foot is estimated between \$90-\$145 (midpoint \$118). The estimates are the total cost and, therefore, the deck itself is likely some fraction. For the purposes of this example a square foot of deck will be assumed to cost \$100.

Of the 40 randomly sampled spans, seven (17.5%) failed to meet the acceptance threshold (the average cover being within 0.25" pre-grinding) and would thus be outright rejected (likely corrective action would be called for). Ideally, the new pay factors will incentivize contractors to reduce the number of bridges which fail to meet the threshold outright. Consequently, all the subsequent analysis was performed on the remaining 33 spans.

Table 18, which includes the data on the 33 bridges provides a few insights. The first of which is that none of the pay factor equations, even the ones that allow for bonus payments, had an average pay factor in excess of 100. This can be said despite a number of spans well in excess of 100% payment. One of the objectives is to have a gradual transition, and therefore the pay factor equation results should have a large standard deviation. In this metric, equations 6 and 7 have a clear advantage over equations 8 and 9. These results are summarized in graphical manner in Figure 31.

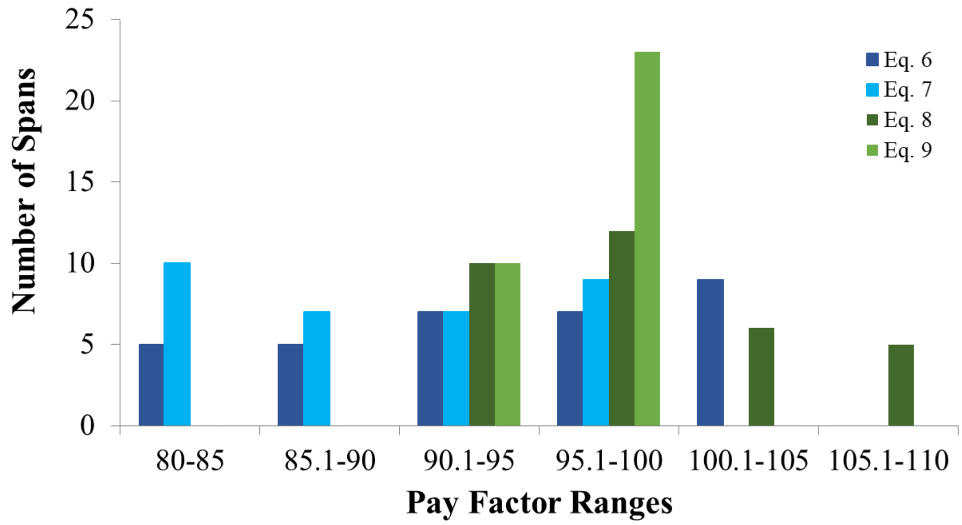


Figure 31. Graph. Histogram showing the proportion of spans within each pay factor range.

Table 18. Summary of results of the 33 spans

Span	Design Cover (in)	Avg. Cover (in)	Std. Dev. (in)	No. of Points	Avg. Cover - Design Cover	PWL	PF Eq. 6	PF Eq. 7	PF Eq. 8	PF Eq. 9
1	2.25	2.23	0.17	16	-0.02	87.0	98.5	93.5	99.1	99.1
2	2.5	2.54	0.35	12	0.04	51.0	80.5	75.5	101.6	100.0
3	2	2.00	0.16	36	0.00	90.0	100.0	95.0	100.0	100.0
4	2.5	2.38	0.32	44	-0.13	53.0	81.5	76.5	95.0	95.0
5	2.75	2.56	0.09	84	-0.19	74.0	92.0	87.0	93.1	93.1
6	2	2.06	0.27	67	0.06	64.0	87.0	82.0	103.0	100.0
7	2.5	2.41	0.15	16	-0.09	85.0	97.5	92.5	96.4	96.4
8	2.25	2.16	0.18	36	-0.09	79.0	94.5	89.5	96.0	96.0
9	2.25	2.10	0.12	10	-0.15	79.0	94.5	89.5	93.3	93.3
10	2.25	2.12	0.11	48	-0.13	86.0	98.0	93.0	94.2	94.2
11	2.25	2.47	0.18	20	0.22	56.0	83.0	78.0	109.8	100.0
12	2.25	2.43	0.13	12	0.18	69.0	89.5	84.5	108.0	100.0
13	2.75	2.60	0.07	12	-0.15	93.0	101.5	96.5	94.5	94.5
14	2.75	2.65	0.19	96	-0.10	75.0	92.5	87.5	96.4	96.4
15	2.5	2.35	0.07	35	-0.15	92.0	101.0	96.0	94.0	94.0
16	2.75	2.78	0.12	42	0.03	97.0	103.5	98.5	101.1	100.0
17	2.5	2.47	0.17	45	-0.03	85.0	97.5	92.5	98.8	98.8

18	2.25	2.43	0.15	12	0.18	67.0	88.5	83.5	108.0	100.0
19	2.5	2.67	0.11	18	0.17	76.0	93.0	88.0	106.8	100.0
20	2.25	2.43	0.07	24	0.18	83.0	96.5	91.5	108.0	100.0
21	2.75	2.63	0.07	45	-0.12	97.0	103.5	98.5	95.6	95.6
22	2.75	2.76	0.09	50	0.01	100.0	105.0	100.0	100.4	100.0
Span	**Design Cover (in)**	**Avg. Cover (in)**	**Std. Dev. (in)**	**No. of Points**	**Avg. Cover - Design Cover**	**PWL**	**PF Eq. 6**	**PF Eq. 7**	**PF Eq. 8**	**PF Eq. 9**
23	2.75	2.74	0.03	75	-0.01	100.0	105.0	100.0	99.6	99.6
24	2.75	2.72	0.16	50	-0.03	88.0	99.0	94.0	98.9	98.9
25	2.75	2.55	0.12	28	-0.20	65.0	87.5	82.5	92.7	92.7
26	2.75	2.56	0.11	8	-0.19	69.0	89.5	84.5	93.1	93.1
27	2.75	2.82	0.13	5	0.07	94.0	102.0	97.0	102.5	100.0
28	2.25	2.26	0.10	100	0.01	100.0	105.0	100.0	100.4	100.0
29	2.25	2.25	0.22	60	0.00	74.0	92.0	87.0	100.0	100.0
30	2.5	2.29	0.17	35	-0.21	59.0	84.5	79.5	91.6	91.6
31	2.75	2.71	0.11	52	-0.04	98.0	104.0	99.0	98.5	98.5
32	2.25	2.15	0.19	131	-0.10	74.0	92.0	87.0	95.6	95.6
33	2.75	2.51	0.16	20	0.24	52.0	81.0	76.0	91.3	91.3
						Average:	94.6	89.6	98.7	97.2
						St Dev:	7.44	7.44	5.07	2.98

From Figure 31, the inadequacies of equations 8 and 9 are more apparent, as there is a sharp peak and narrow payment band, in direct contradiction to the desired pay factor structure. Amongst equations 6 and 7, the lack of the bonus in Equation 7 causes a shift in the prevalence of pay factors to the left, toward lower average payments, which has the negative effect of doubling the number of spans paid at the 80-85% ranges compared to Equation 6.

Considering equations from a cost perspective, with the assumption that each cover measurement was approximately 10 ft from its nearest neighbor (in a square grid), each data point has a tributary area of approximately 100 square feet (half the distance to neighbor squared). Therefore, the preadjustment cost for a bridge deck is estimated by Equation 10, *w*here *n* is the number of sampled points in the cover survey. The results from applying the payment factors to the estimated deck costs can be seen in Table 19.

$$Preadjustment\ Deck\ Cost = n * 100\ ft^2 * \$100/ft^2 \qquad \text{Eq. 10}$$

Table 19 provides significant insights as overall trends, without focusing on the specific values. Without exception, all the proposed pay factor equations result in a net reduction in the overall payment for this set of 33 spans, which is expected given the sub 100% average pay factors in Table 18. That trend may not be desirable because the overall industry would expect lower prices, which may just result in an industry wide increase in bid prices as opposed to improvements in their construction practices.

As noted in (Novak et al 2018), to combat this possibility, the pay factor equation was altered to result in a net overpayment of 3%. The negative trend is more pronounced in the pay factor equations without the ability to exceed 100% for the pay, which as mentioned prior, helps to offset the risk that the sampling causes a bridge to receive less payment than the work may be entitled to, as well as providing an economic incentive to improve. Between equations 6 and 7, the lack of a bonus causes the average and net losses over the entire series of spans to double. Interestingly, between equations 8 and 9, there is not a doubling of the average and net losses, but rather an approximately 39% increase. Equation 6 provides the most diverse spread between the best and worst spans evaluated, providing the largest reward (~$50k) and the second largest penalty (~$105k), which would be ideal for incentivizing improvements.

Table 19. Estimates of the economic consequences of the proposed pay equations

	Gain or Loss in Payment			
Estimated Deck Cost	**Eq. 6**	**Eq. 7**	**Eq. 8**	**Eq. 9**
$160,000	-$2,400	-$10,400	-$1,422	-$1,422
$120,000	-$23,400	-$29,400	$1,920	$0
$360,000	$0	-$18,000	$0	$0
$440,000	-$81,400	-$103,400	-$22,000	-$22,000
$840,000	-$67,200	-$109,200	-$58,036	-$58,036
$670,000	-$87,100	-$120,600	$20,100	$0
$160,000	-$4,000	-$12,000	-$5,760	-$5,760
$360,000	-$19,800	-$37,800	-$14,400	-$14,400
$100,000	-$5,500	-$10,500	-$6,667	-$6,667
$480,000	-$9,600	-$33,600	-$27,733	-$27,733
$200,000	-$34,000	-$44,000	$19,556	$0
$120,000	-$12,600	-$18,600	$9,600	$0
$120,000	$1,800	-$4,200	-$6,545	-$6,545
$960,000	-$72,000	-$120,000	-$34,909	-$34,909
$350,000	$3,500	-$14,000	-$21,000	-$21,000
$420,000	$14,700	-$6,300	$4,582	$0
$450,000	-$11,250	-$33,750	-$5,400	-$5,400
$120,000	-$13,800	-$19,800	$9,600	$0
$180,000	-$12,600	-$21,600	$12,240	$0
$240,000	-$8,400	-$20,400	$19,200	$0
$450,000	$15,750	-$6,750	-$19,636	-$19,636
$500,000	$25,000	$0	$1,818	$0
$750,000	$37,500	$0	-$2,727	-$2,727
$500,000	-$5,000	-$30,000	-$5,455	-$5,455
$280,000	-$35,000	-$49,000	-$20,364	-$20,364
$80,000	-$8,400	-$12,400	-$5,527	-$5,527
$50,000	$1,000	-$1,500	$1,273	$0
$1,000,000	$50,000	$0	$4,444	$0
$600,000	-$48,000	-$78,000	$0	$0
$350,000	-$54,250	-$71,750	-$29,400	-$29,400
$520,000	$20,800	-$5,200	-$7,564	-$7,564
$1,310,000	-$104,800	-$170,300	-$58,222	-$58,222
$200,000	-$38,000	-$48,000	-$17,455	-$17,455
Average:	-$17,832	-$38,195	-$8,057	-$11,219
Net:	-$588,450	-$1,260,450	-$265,890	-$370,223
Max:	$50,000	$0	$20,100	$0
Min:	-$104,800	-$170,300	-$58,222	-$58,222

Considering the results from Table 18 and Table 19 as well as Figure 31, it is in the opinion of the authors that Equation 6 is best suited for purposes of improving the cover construction practices. However there still exists uncertainty with the discrepancies between the sampled cover distribution and the true cover distribution. To investigate these risks, the software OCPLOT was utilized to examine Equation 6 (Weed 1995). As part of the analysis, OCPLOT creates 500 random sampling trials (of the specified sampling number, in this case n=20) on a simulated bridge deck with a specified "true" PWL and standard deviation. The software determines the sampled PWL and the corresponding pay factor for each trial. The software then uses the results from the aggregate of the trials to determine the expected PWL and the expected pay factor according to Equation 6. This allows the user to assess the risks associated with the pay factor equation, which can then be altered if needed. The results from that evaluation are given in the curve in Figure 32.

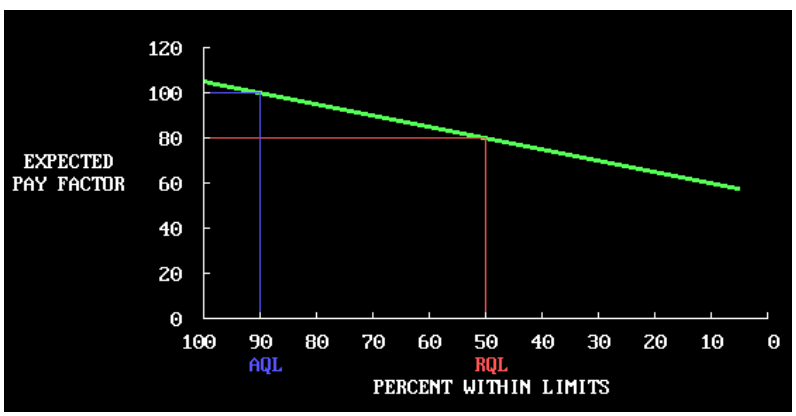


Figure 32. Graph. Output from OCPLOT showing the expected pay factor given the percent within the specification limits.

Figure 32 displays graphically the results expected when using Equation 6 for determining the payment. The figure supports the notion that if the work submitted is exactly at the adequate quality level, then it is expected that you will receive 100% of the pay even though for any one span the sampling will cause the contractor to receive too much or too little pay. To demonstrate how each sampling affects the pay factor, examine Figure 33, which shows that if the work was truly 50 PWL, the varaibility in the estimated PWL from the 20 sampled points (min/max of ~25 PWL and 78 PWL) and how that affects the corresponding pay factor.

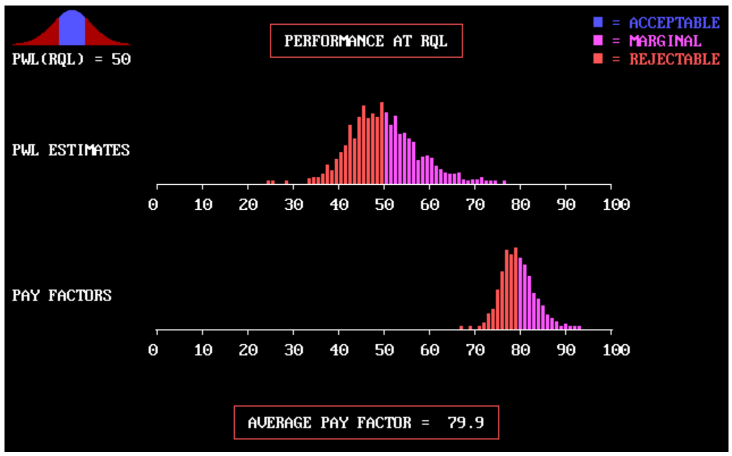


Figure 33. Graph. OCPLOT output demonstrated the range in PWL estimated from sampling and corresponding range in payment.

From Figure 33, on average, a contractor could expect a pay factor of 80% given work that is 50 PWL with the proposed sampling scheme. With fewer random samples the spread in PWL estimates is expected to increase, which may increase the risk that on any one span evaluation that the work receives an inaccurate pay factor. Therefore the proposed specification, as provided in Appendix D, incorporates Equation 6 as well as the random sampling procedure (n=20).

6. CONCLUSIONS AND RECOMMENDATIONS

The main conclusions and recommendations from the research project are as follows:

1. Cover depth that is too deep leads to cracking in the decks, which then also contributes to degradation by corrosion, salt scaling, and/or freeze/thaw cycling. Premature degradation results in the necessity for more frequent inspections, earlier and additional maintenance and repair, and eventually, reconstruction. From interviews, GDOT personnel identified the likely causes of poor cover control to be the following: contractors intentionally pouring too much cover in anticipation of surface grinding later, failed formwork, improper placement of the screed rails, unsupported rebar mats (particularly near the headers), and general human error. Poor cover control is typically remediated through washing out the unset concrete, hydrodemolishing the deck when the concrete is plastic, or applying an overlay (i.e., epoxy).
2. Based on an analysis of GDOT historical records, there does not appear to be a trend toward excessive or insufficient cover from the late 1970s to present, with greater than 90% of bridges having an average sampled cover within 0.25 inch of the design cover. It appears that poor cover control can be either random or systematic in form, as determined through two-dimensional cover mapping for select bridges. On average, data indicate that 40% of the deck area is below the specified design cover, while 60% is above. Approximately 20% of the deck area, on average, is more than 0.5 inch below the specification. The cover distribution for an average bridge with a 2.75-inch cover may be approximated by a normal distribution with mean of 2.68 inches and a standard deviation of 0.13 inch. These

results are based on data points sampled within a ten-foot grid. GDOT should further investigate the relationship between the cover sampling grid size and the actual cover distribution of bridge decks. The results from this investigation could be used to draft a cover measurement spacing specification for the plastic concrete evaluations, which would strengthen contracting mechanisms.

3. On average, for a given decommissioned bridge that was investigated, the substructure had the lowest National Bridge Inventory (NBI) rating while the deck had the highest. More than 60% of the decommissioned bridge decks investigated had some form of spalling, which depending on the location and severity could severely disrupt the ability of traffic to safely pass. Based on the high frequency of spalling, with or without exposed rebar, a corrosion model should provide meaningful insights into the degradation of Georgia bridge decks.
4. From the probabilistic and one-dimensional simulations developed, the average expected decrease in service life is approximately two years per 0.1 inch of insufficient cover. While minor variations of a couple of years on the service life may not be significant for any one bridge, over the entire inventory small losses may aggregate to a significant effect. Because of this significant aggregated effect, GDOT should consider tightening the tolerance for cover control in the bridge construction manual, to ensure longer service lives for bridge decks.
5. Based on a case study using Life-365™, a larger increase in service life is expected from the incorporation of fly ash as opposed to the transition between mix design classes as GDOT currently intends (from Class A to Class D concrete for bridge decks). As cement is the most expensive constituent of concrete, it may also be

more cost effective to maintain the existing mix class with the addition of fly ash. GDOT should consider adding a greater amount of supplementary cementing materials to the default concrete mixes to promote stronger and more robust concrete bridge decks.

6. The Georgia Code only mentions liquidated damages in relations to public works projects. The most relevant statutes can be found in §13-10-70 of the Georgia Code: "Liquidate damages for late completion and incentives for early completion." The state codes and statutes of Virginia, Texas, Indiana, Florida, Ohio, Utah, and California were also examined. In general, it appears that the states defer to the Universal Commercial Code (UCC), in whole or with modification, to serve as the general basis for their contracting laws, with specific amendments by statute. The overall consensus is that there may be no penalty clauses in contracts without the prospect of receiving a bonus. The cases Southeastern Land Fund v. Real Estate World and Fortune Bridge Co. v. Department of Transportation that were tried before the Supreme Court of Georgia affirm the use of liquidated damages for cases that meet specific legal requirements. It appears that a liquidated damages clause may be enforceable for cover deficiencies if GDOT were unable to accurately determine the actual damages in advance, and that other legal remedies for breach are still available if explicitly mentioned in the contract.
7. For this particular application, namely cover control, Incentive/Disincentive provisions such as the examples given are not well suited to ensuring cover control as GDOT intends to improve construction quality as opposed to construction speed or cost. It would also be unreasonable to expect a contractor to warranty a bridge

deck for its entire service life based on its cover control. Doing so may be very costly to GDOT. For this reason, warranties may not be a good option for cover control. Additionally, if GDOT were to transition to a Design-Build-Operate-Maintain (DBOM) framework for future construction, the impacts of insufficient cover may be borne by the contractor, who will have an incentive to build a quality bridge to avoid that liability. There are, however, general policy and legal concerns that will need to be addressed before DBOM frameworks become widely adopted for bridge construction.

8. The research found that the methodologies from existing GDOT methods such as GDT 73 Method C or SOP 46 could be readily adapted to cover control to develop an acceptance/adjustable payment plan. GDOT should consider adopting contracting language that incorporates an acceptance/adjustable payment plan such as the one proposed in this study to incentivize proper cover control.

APPENDICES

1. APPENDIX A: SELECT COVER SURVEYS

D.O.T. 66

C.G.

DEPARTMENT OF TRANSPORTATION
STATE OF GEORGIA

INTERDEPARTMENT CORRESPONDENCE

FILE ID-85-2 (92) Fulton
Lunda Construction Co.
Br. Nos. 33 & 36

OFFICE Materials & Research
Forest Park, Georgia

DATE November 20, 1981

FROM Tom Stapler, P.E., State Materials and Research Engineer

TO Larry Adams, District Engineer, Atlanta

SUBJECT STEEL COVER (BRIDGE DECKS)
PRESTRESSED CONCRETE GIRDER

Pachometer measurements to determine the concrete cover over the top reinforcing steel on the referenced project have been obtained.

Listed below is a summary of these measurements.

BRIDGE NO.	SPAN OR POUR NO.	DESIGN COVER	AVERAGE COVER	RANGE (IN.)	COEFFICIENT OF VARIATION
33	1	2 1/2	2.42	1 1/2 - 3	16.7
33	2	2 1/2	1.87	5/8 - 2 3/4	26.0
33	3	2 1/2	2.63	2 1/8 - 3	8.4
36	1	2 1/2	2.52	2 1/4 - 2 3/4	6.2
36	2	2 1/2	1.93	1 1/4 - 2 3/4	16.8
36	3	2 1/2	2.35	1 3/4 - 2 3/4	11.0

These measurements indicate problems with Span 2 on both bridges. A small area of the top mat is exposed on Span 2, Bridge No. 33. Cores were obtained from these two spans. The measrements obtained are on the attached drawings. Attached also are the distribution charts showing individual measurements and other related data to steel cover.

If we may be of further assistance, please let me know.

TS/CGB/es
Attachment
c: Hal Rives
Alva Byrom
Wendell Lawing
Lunda Construction Co.

DECK STEEL COVER

Distribution Chart

CONTRACTOR: Lunda Construction Company

PROJECT NO. ID-85-2 (92) Fulton

BRIDGE NO. 33 SBL I-85/Peachtree Cr. Relocation

SPAN OR POUR NO. 1

TYPE STRUCTURE: Prestressed Concrete Girders

RATING: - C

- 0.49	
0.50 - 0.75	
0.76 - 1.00	
1.01 - 1.25	
1.26 - 1.50	1
1.51 - 1.75	
1.76 - 2.00	5
2.01 - 2.25	4
2.26 - 2.50	8
2.51 - 2.75	7
2.76 - 3.00	4
3.01 -	
Total No. of Readings:	30
Range:	1½ - 3

Design Cover 2½

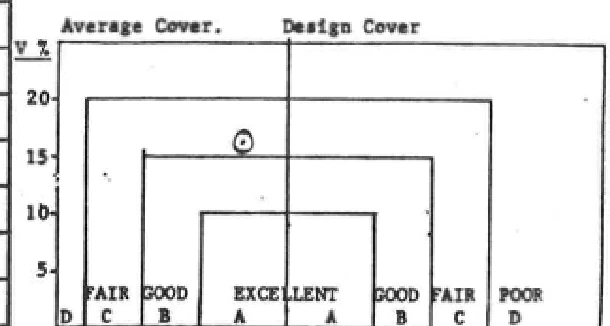
Average Cover ($\bar{X}$) 2.42

Standard Deviation (S) .41

Coefficient of Variation (V) 16.7

RATING DIAGRAM

Used to determine Contractor performance based on Coefficient of Variation and Average Cover.



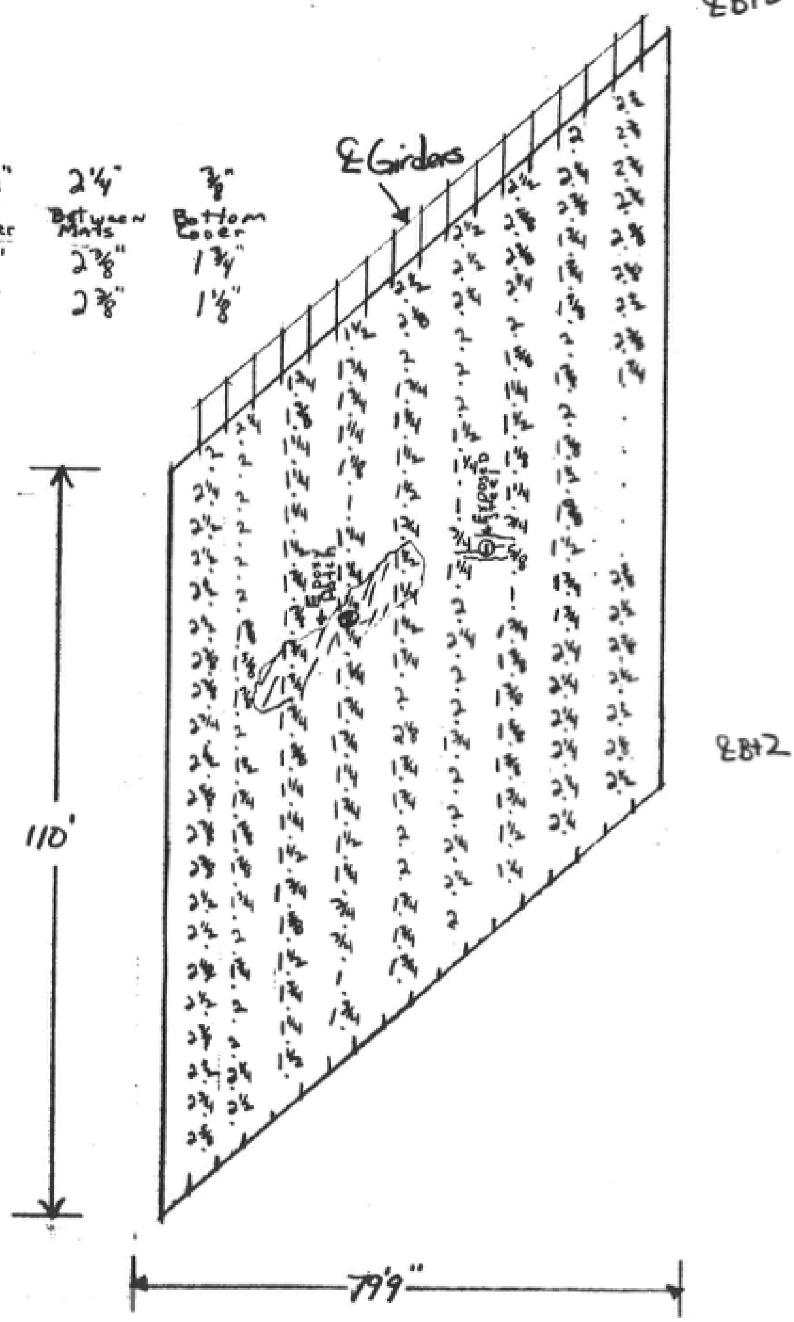
Average Cover for	2" Design -----	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5
" " " "	2½" Design-----	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3.0

INDIVIDUAL MEASUREMENTS

in.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0								
1					2			
2	5	1	3	2	6	2	5	
3	4							

Bridge #33
Span 2

Core No.	Slab Thickness	Top Cover	Between Mats	Bottom Cover
Theo	7"	2½"	2¼"	⅞"
1	6⅞"	0"	2⅜"	1¾"
2	6½	1"	2⅜"	1⅛"



Scale: 1" = 20'

DECK STEEL COVER

Distributi... Chart

CONTRACTOR: Lunda Construction Company

PROJECT NO. ID-85-2 (92) Fulton

BRIDGE NO. 33 SBL I-85/Peachtree Cr. Relocation

SPAN OR POUR NO. 2

TYPE STRUCTURE: Prestressed Concrete Girders

RATING: - off diagram

- 0.49	
0.50 - 0.75	5
0.76 - 1.00	4
1.01 - 1.25	23
1.26 - 1.50	19
1.51 - 1.75	38
1.76 - 2.00	36
2.01 - 2.25	17
2.26 - 2.50	31
2.51 - 2.75	11
2.76 - 3.00	
3.01 -	
Total No. of Readings:	184
Range:	5/8 - 2 3/4

Design Cover $2\frac{1}{2}$

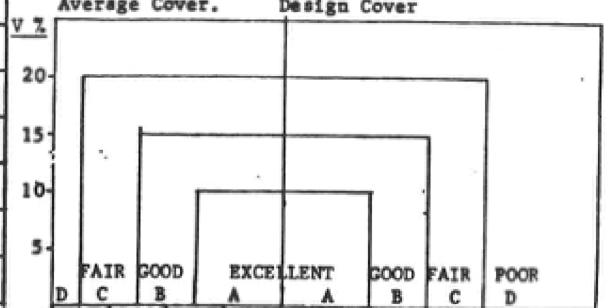
Average Cover ($\bar{X}$) 1.87

Standard Deviation (S) .49

Coefficient of Variation (V) 26.0

RATING DIAGRAM

Used to determine Contractor performance based on Coefficient of Variation and Average Cover.



Average Cover for										
2" Design -----	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5
$2\frac{1}{2}$" Design-----	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3.0

INDIVIDUAL MEASUREMENTS

in.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0						1	4	
1	4	2	21	3	16	6	32	9
2	27	3	14	9	22	6	5	
3								
4								

DECK STEEL COVER

Distribution Chart

CONTRACTOR: Lunda Construction Company

PROJECT NO. ID-85-2 (92) Fulton

BRIDGE NO. 36 SBL I-85/Lindberg Dr.

SPAN OR POUR NO. 1

TYPE STRUCTURE: Prestressed Concrete Girders

RATING: - A

- 0.49	
0.50 - 0.75	
0.76 - 1.00	
1.01 - 1.25	
1.26 - 1.50	
1.51 - 1.75	
1.76 - 2.00	
2.01 - 2.25	2
2.26 - 2.50	9
2.51 - 2.75	4
2.76 - 3.00	
3.01 -	
Total No. of Readings:	15
Range:	2 1/4 - 2 3/4

Design Cover: 2½

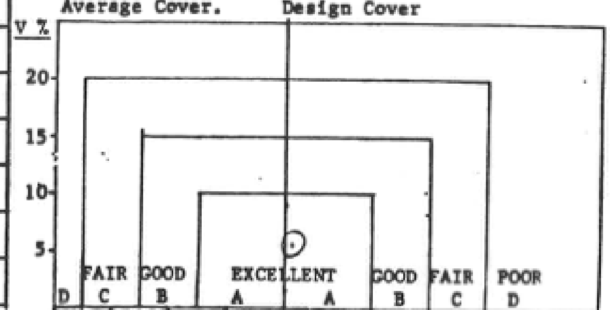
Average Cover ($\bar{X}$): 2.52

Standard Deviation (S): .16

Coefficient of Variation (V): 6.2

RATING DIAGRAM

Used to determine Contractor performance based on Coefficient of Variation and Average Cover.



Average Cover for										
2" Design -----	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5
2½" Design-----	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3.0

INDIVIDUAL MEASUREMENTS

in.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0								
1								
2			2	1	8	1	3	
3								
4								

DECK STEEL COVER

Distribution Chart

CONTRACTOR: Lunda Construction Company

PROJECT NO. ID-85-2 (92) Fulton

BRIDGE NO. 33 SBL I-85/Peachtree Cr. Relocation

SPAN OR POUR NO. 3

TYPE STRUCTURE: Prestressed Concrete Girders

RATING: - A

- 0.49	
0.50 - 0.75	
0.76 - 1.00	
1.01 - 1.25	
1.26 - 1.50	
1.51 - 1.75	
1.76 - 2.00	
2.01 - 2.25	3
2.26 - 2.50	7
2.51 - 2.75	16
2.76 - 3.00	4
3.01 -	
Total No. of Readings:	30

Range: 2 1/8 - 3

Design Cover 2½

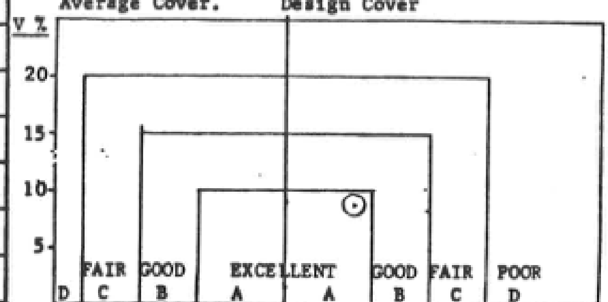
Average Cover ($\bar{X}$) 2.63

Standard Deviation (S) .22

Coefficient of Variation (V) 8.4

RATING DIAGRAM

Used to determine Contractor performance based on Coefficient of Variation and Average Cover.



Average Cover for 2" Design ----- 1.6 1.7 1.8 1.9 2.0 2.1 2.2 2.3 2.4 2.5
" " " 2½" Design----- 2.1 2.2 2.3 2.4 2.5 2.6 2.7 2.8 2.9 3.0

INDIVIDUAL MEASUREMENTS

in.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0								
1								
2		2	1	1	6	6	10	1
3	3							
4								

DECK STEEL COVER

Distribution Chart

CONTRACTOR: Lunda Construction Company

PROJECT NO. ID-85-2 (92) Fulton

BRIDGE NO. 36 SBL I-85 Lindberg Dr.

SPAN OR POUR NO. 2

TYPE STRUCTURE: Prestressed Concrete Girders

RATING: - off diagram

- 0.49	
0.50 - 0.75	
0.76 - 1.00	
1.01 - 1.25	13
1.26 - 1.50	21
1.51 - 1.75	49
1.76 - 2.00	60
2.01 - 2.25	53
2.26 - 2.50	21
2.51 - 2.75	3
2.76 - 3.00	
3.01 -	
Total No. of Readings:	220

Range: 1 1/4 - 2 3/4

Design Cover 2½

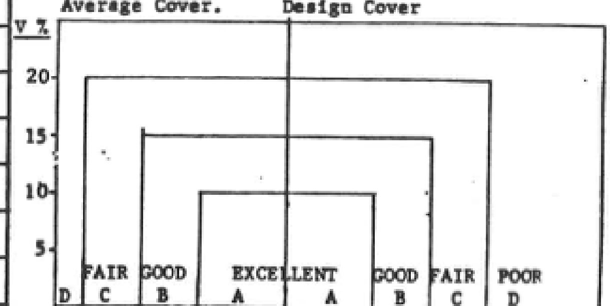
Average Cover ($\bar{X}$) 1.93

Standard Deviation (S) .33

Coefficient of Variation (V) 16.8

RATING DIAGRAM

Used to determine Contractor performance based on Coefficient of Variation and Average Cover.



Average Cover for 2" Design ----- 1.6 1.7 1.8 1.9 2.0 2.1 2.2 2.3 2.4 2.5
" " " 2½" Design----- 2.1 2.2 2.3 2.4 2.5 2.6 2.7 2.8 2.9 3.0

INDIVIDUAL MEASUREMENTS

in.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0								
1			13	6	15	13	36	27
2	33	19	34	11	10	2	1	
3								
4								

DECK STEEL COVER

Distribution Chart

CONTRACTOR: Lunda Construction Company

PROJECT NO. ID-85-2 (92) Fulton

BRIDGE NO. 36

SPAN OR POUR NO. 3

TYPE STRUCTURE: Prestressed Concrete Girders

RATING: - B

- 0.49	
0.50 - 0.75	
0.76 - 1.00	
1.01 - 1.25	
1.26 - 1.50	
1.51 - 1.75	1
1.76 - 2.00	3
2.01 - 2.25	5
2.26 - 2.50	8
2.51 - 2.75	3
2.76 - 3.00	
3.01 -	
Total No. of Readings:	20
Range:	1 3/4 - 2 3/4

Design Cover 2½

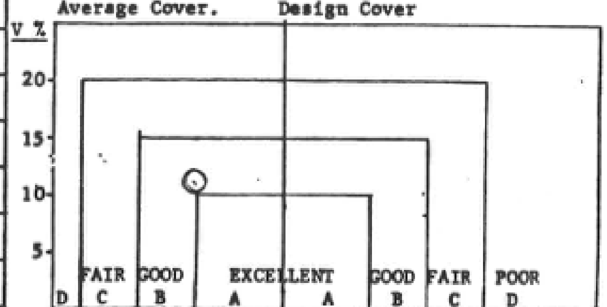
Average Cover ($\bar{X}$) 2.35

Standard Deviation (S) .26

Coefficient of Variation (V) 11.0

RATING DIAGRAM

Used to determine Contractor performance based on Coefficient of Variation and Average Cover.



Average Cover for 2" Design ----- 1.6 1.7 1.8 1.9 2.0 2.1 2.2 2.3 2.4 2.5

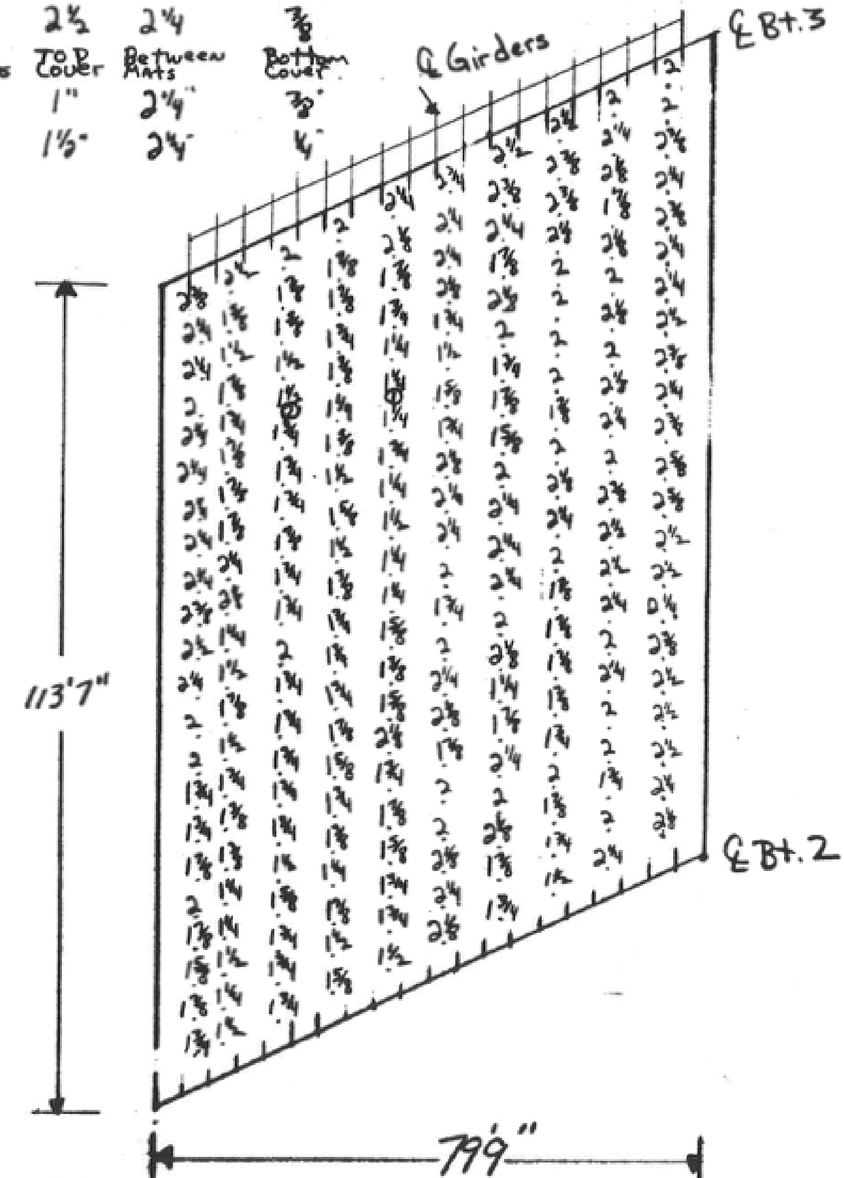
" " " " 2½" Design----- 2.1 2.2 2.3 2.4 2.5 2.6 2.7 2.8 2.9 3.0

INDIVIDUAL MEASUREMENTS

in.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0								
1							1	
2	3		5		8	2	1	
3								
4								

Bridge #36
Span 2

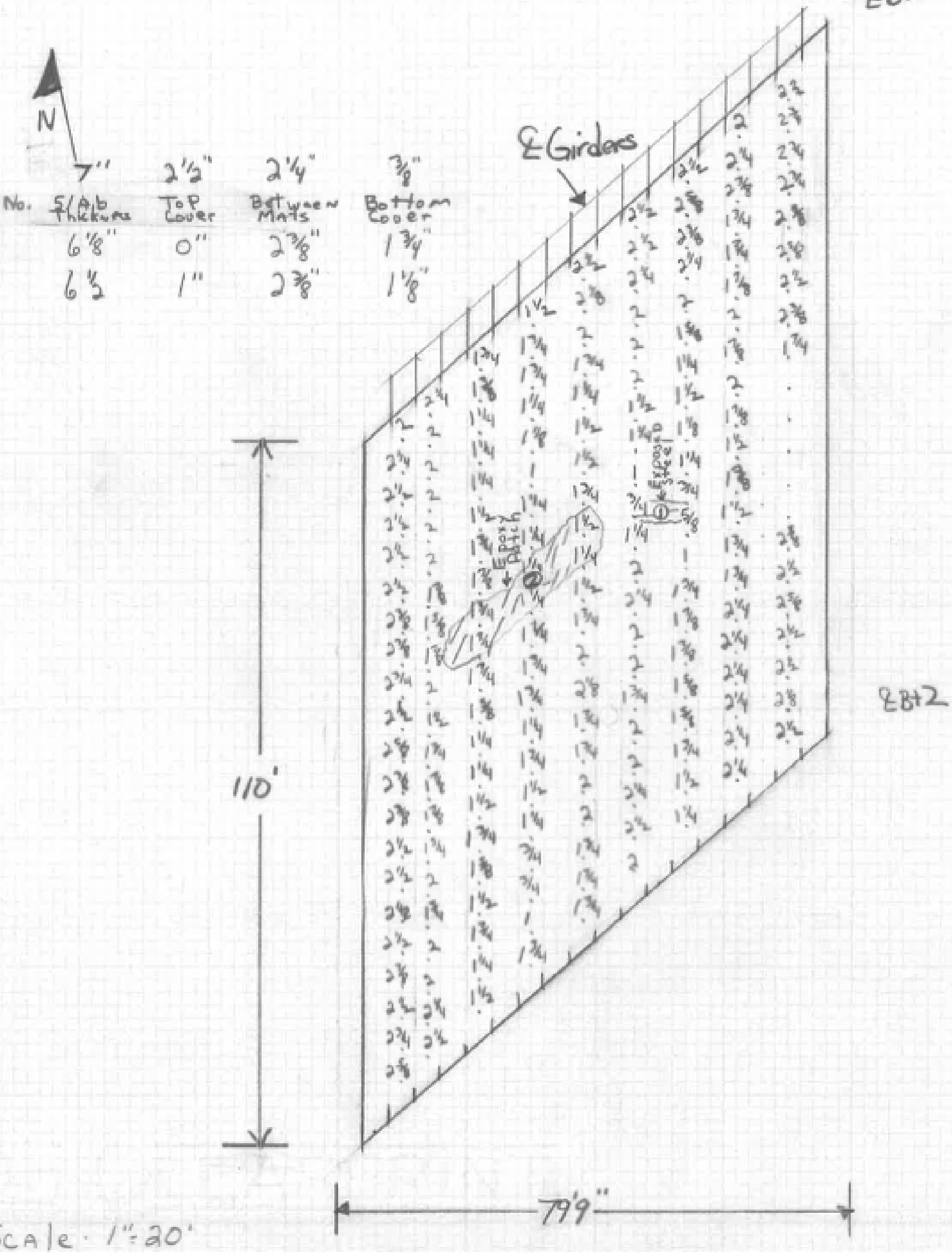
Core No.	Slab Thickness	Top Cover	Between Mats	Bottom Cover
Theo.	7"	2 1/2	2 1/4	3/8
1	5 5/8"	1"	2 1/4"	3/8"
2	6"	1 1/2"	2 1/4"	1/4"



Scale 1" = 20'

Bridge #33
Span 2

Core No.	Slab Thickness	Top Cover	Between Mats	Bottom Cover
Theo	7"	2 1/2"	2 1/4"	3/8"
1	6 7/8"	0"	2 3/8"	1 3/4"
2	6 1/2"	1"	2 3/8"	1 1/8"

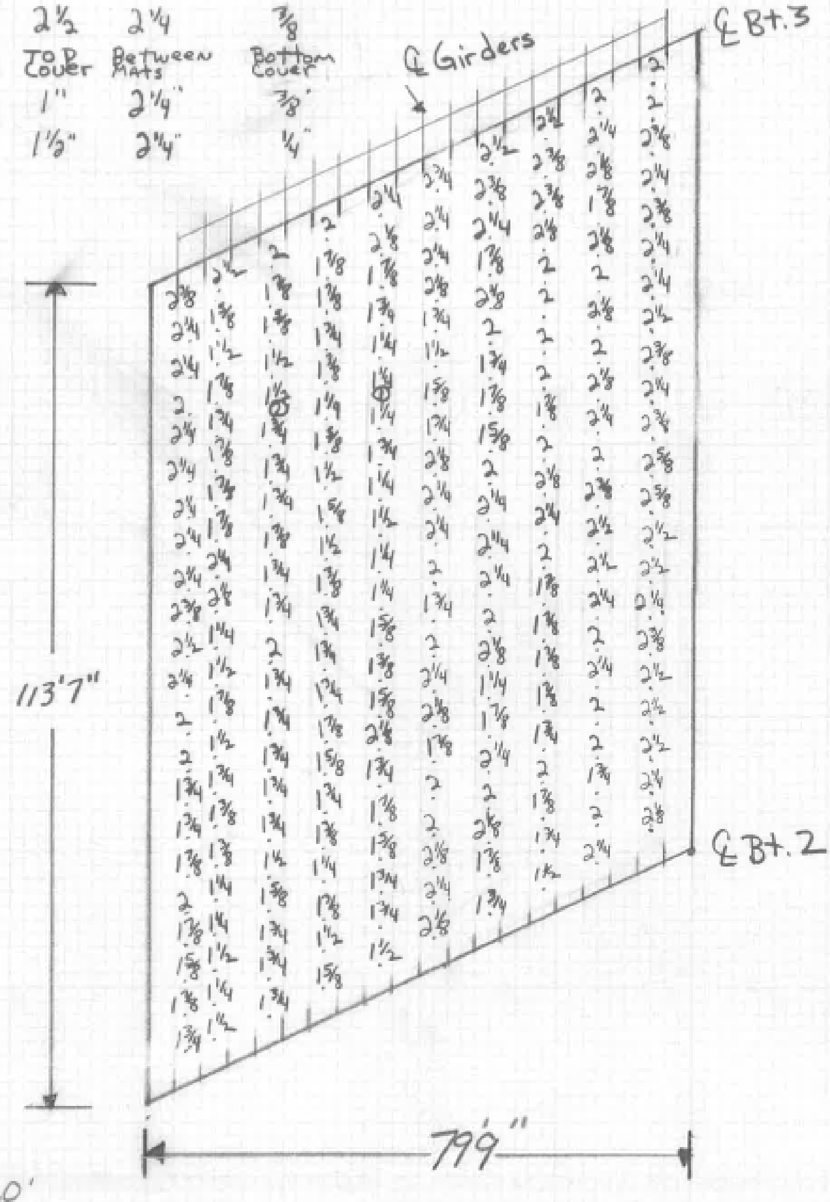


Scale 1" = 20'

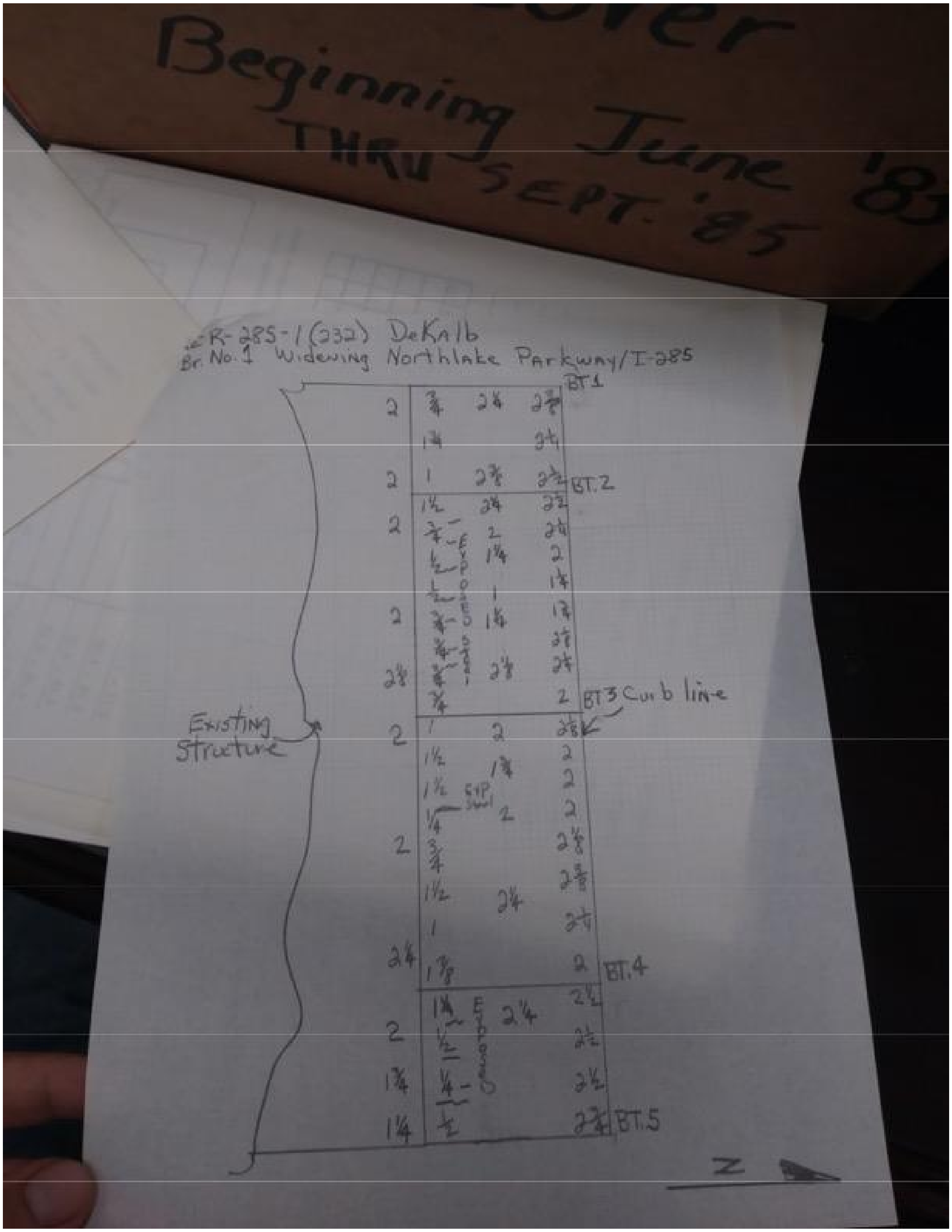
Bridge #36
Span 2

N

Core No.	Slab Thickness	Top Cover	Between Mats	Bottom Cover
Theo.	7"	2 1/2	2 1/4	3/8
1	5 5/8"	1"	2 1/4"	3/8"
2	6"	1 1/2"	2 1/4"	1/4"



Scale 1" = 20'


Beginning June '8
THRU SEPT. '85
R-285-1(232) DeKalb
Br. No. 1 Widening Northlake Parkway/I-285
BT.1
BT.2
BT.3 Curb line
Existing Structure
BT.4
BT.5
N

Beginning June 85
THRU SEPT '85

Steel Cover

D.O.T. 88

DEPARTMENT OF TRANSPORTATION
STATE OF GEORGIA

INTERDEPARTMENT CORRESPONDENCE

FILE IR-285-1(232) Dekalb
Br. No. 1, Widening Northlake Parkway/I-285

OFFICE Materials and Research
Forest Park

DATE March 28, 1985

FROM Tom Stapler, State Materials and Research Engineer

TO John W. Wade, Jr., District Engineer, Chamblee

SUBJECT STEEL COVER (BRIDGE DECKS)
STEEL GIRDER CONSTRUCTION

Pachometer measurements to determine the concrete cover over the top reinforcing steel on the referenced project have been obtained.

Listed below is a summary of these measurements:

BRIDGE NO.	SPAN OR POUR NO.	DESIGN COVER	AVERAGE COVER	RANGE (IN.)	COEFFICIENT OF VARIATION
1	1-4	2 3/4	1.64"	1/4 - 2 3/4	42.9

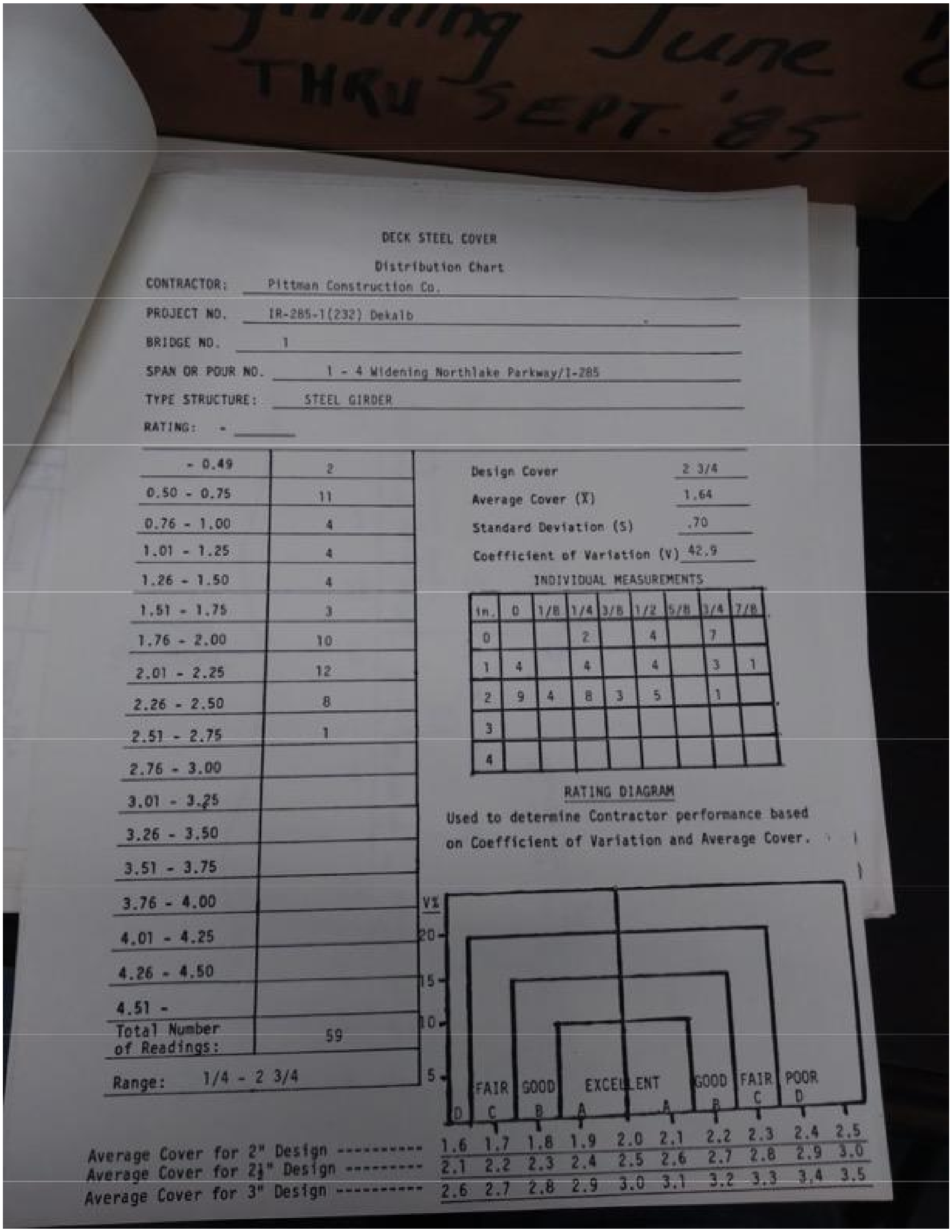
Cover readings obtained indicated unacceptable top mat cover in all spans. The portion of the new construction along the existing structure is the area most deficient in cover. Steel is exposed in Spans 2, 3, and 4. This exposed steel is the existing reinforcement that wasn't bent down to the proper location in the new construction. Random cover checks made on the old structure indicated sufficient steel cover.

Attached are distribution charts and/or drawings showing individual measurements and other related data to steel cover.

If we may be of further assistance, please let me know.

TS/CGB/lt
attachment
c: Alva Byrom
Ronald Edmondson
Charles Lewis
Charles Henderson
Pittman Construction Co.

"MAKE DOT BETTER"



DECK STEEL COVER

Distribution Chart

CONTRACTOR: Pittman Construction Co.

PROJECT NO. IR-285-1(232) Dekalb

BRIDGE NO. 1

SPAN OR POUR NO. 1 - 4 Widening Northlake Parkway/I-285

TYPE STRUCTURE: STEEL GIRDER

RATING: - ______

- 0.49	2
0.50 - 0.75	11
0.76 - 1.00	4
1.01 - 1.25	4
1.26 - 1.50	4
1.51 - 1.75	3
1.76 - 2.00	10
2.01 - 2.25	12
2.26 - 2.50	8
2.51 - 2.75	1
2.76 - 3.00	
3.01 - 3.25	
3.26 - 3.50	
3.51 - 3.75	
3.76 - 4.00	
4.01 - 4.25	
4.26 - 4.50	
4.51 -	
Total Number of Readings:	59

Range: 1/4 - 2 3/4

Design Cover 2 3/4

Average Cover ($\bar{X}$) 1.64

Standard Deviation (S) .70

Coefficient of Variation (V) 42.9

INDIVIDUAL MEASUREMENTS

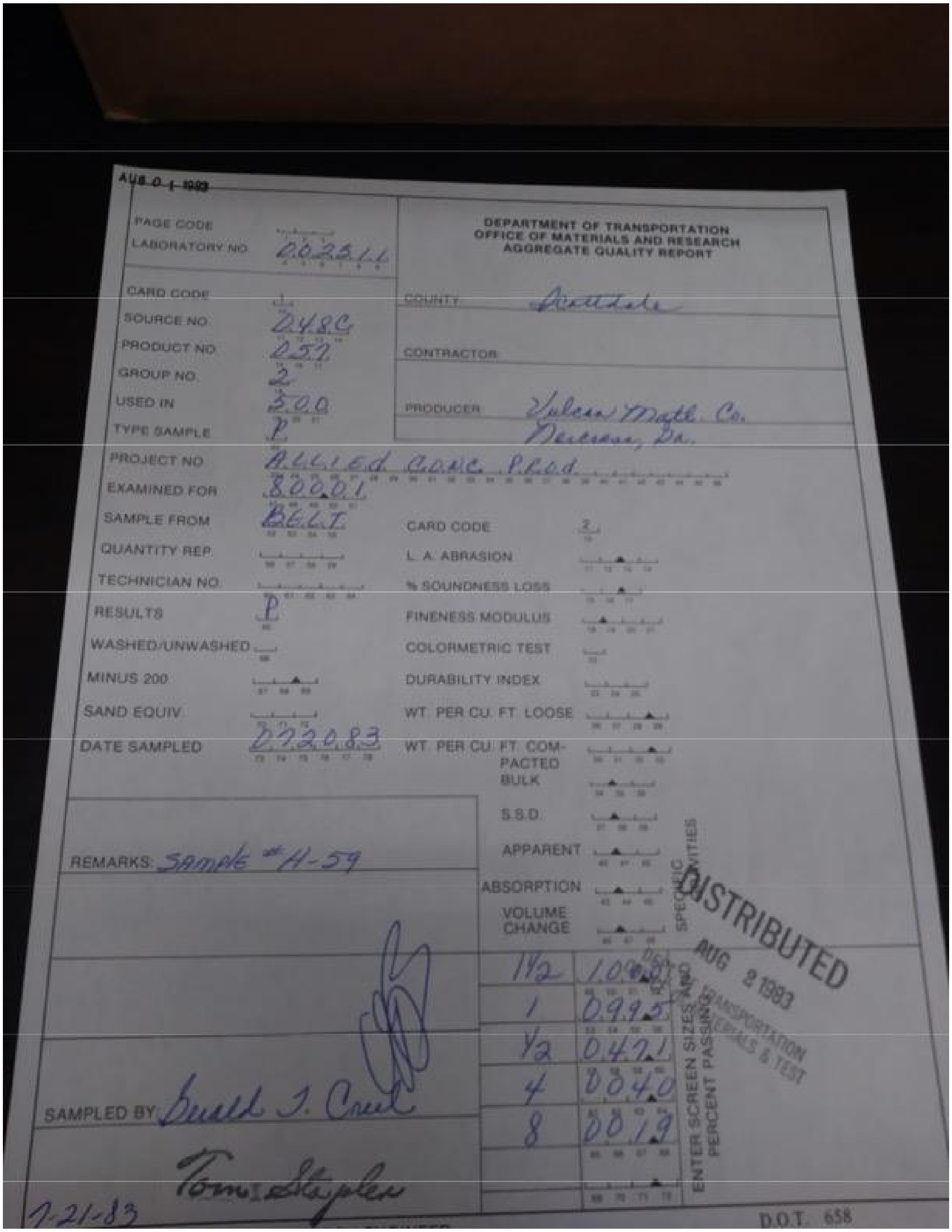
in.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0			2		4		7	
1	4		4		4		3	1
2	9	4	8	3	5		1	
3								
4								

RATING DIAGRAM

Used to determine Contractor performance based on Coefficient of Variation and Average Cover.

V%
20
15
10
5
D
FAIR C
GOOD B
EXCELLENT A
EXCELLENT A
GOOD B
FAIR C
POOR D

Average Cover for 2" Design ----------	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5
Average Cover for 2½" Design ---------	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3.0
Average Cover for 3" Design ----------	2.6	2.7	2.8	2.9	3.0	3.1	3.2	3.3	3.4	3.5



AUG 0 1 1983

PAGE CODE

LABORATORY NO. 002511

DEPARTMENT OF TRANSPORTATION
OFFICE OF MATERIALS AND RESEARCH
AGGREGATE QUALITY REPORT

CARD CODE	1	COUNTY	Scottdale
SOURCE NO.	248C		
PRODUCT NO.	057	CONTRACTOR	
GROUP NO.	2		
USED IN	500	PRODUCER	Vulcan Matl. Co. Decarre, Ga.
TYPE SAMPLE	P		
PROJECT NO.	Allied Conc. Prod.		
EXAMINED FOR	80001		
SAMPLE FROM	BELT	CARD CODE	2
QUANTITY REP.		L. A. ABRASION	
TECHNICIAN NO.		% SOUNDNESS LOSS	
RESULTS	P	FINENESS MODULUS	
WASHED/UNWASHED		COLORMETRIC TEST	
MINUS 200		DURABILITY INDEX	
SAND EQUIV.		WT. PER CU. FT. LOOSE	
DATE SAMPLED	072083	WT. PER CU. FT. COMPACTED	

SPECIFIC GRAVITIES: BULK, S.S.D., APPARENT, ABSORPTION, VOLUME CHANGE

REMARKS: Sample #H-59

ENTER SCREEN SIZES AND PERCENT PASSING

1½	1000
1	0995
½	0471
4	0040
8	0019

DISTRIBUTED
AUG 2 1983
DEPT. OF TRANSPORTATION
MATERIALS & TEST

SAMPLED BY: Buell T. Creel

Tom Stapler

7-21-83

D.O.T. 658

DEPARTMENT OF TRANSPORTATION
STATE OF GEORGIA

INTERDEPARTMENT CORRESPONDENCE

FILE BRZLB-077(1) · COWETA
BRIDGE #1

OFFICE Materials and Research
Forest Park

DATE 11-10-88

FROM PETE MALPHURS, State Materials and Research Engineer

TO BENJAMIN F. ROGERS, AREA ENGINEER, LAGRANGE

SUBJECT STEEL COVER (BRIDGE DECKS)

Pachometer measurements to determine the concrete cover over the top reinforcing steel on the referenced project have been obtained.

Listed below is a summary of these measurements:

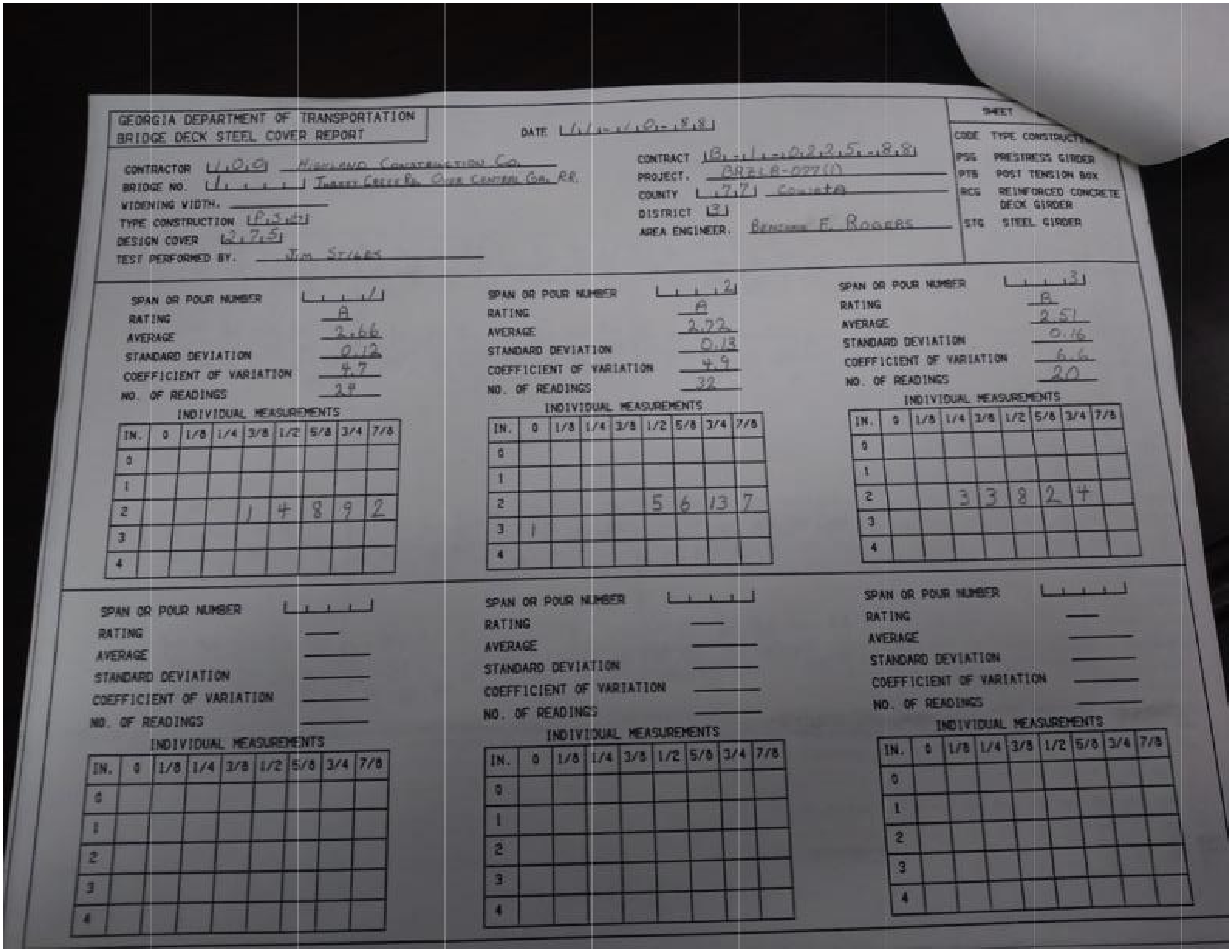
BRIDGE NO.	SPAN OR POUR NO.	DESIGN COVER	AVERAGE COVER	RANGE (IN.)	COEFFICIENT OF VARIATION
1	1	2 3/4	2.66	2 3/8-2 7/8	4.7
1	2	2 3/4	2.72	2 1/2-3	4.9
1	3	2 3/4	2.51	2 1/4-2 3/4	6.6

Attached are distribution charts and/or drawings showing individual measurements and other related data to steel cover.

If we may be of further assistance, please let me know.

TS/
attachment
c: Stanley Lord
DON WATSON
PAUL LILES
BOBBY C. MELTON
HIGHLAND CONSTRUCTION CO.

"MAKE DOT BETTER"


GEORGIA DEPARTMENT OF TRANSPORTATION
BRIDGE DECK STEEL COVER REPORT

DATE 1/1/-/1/0-1/8/8

CONTRACTOR 1,0,0 Highland Construction Co.
BRIDGE NO. Turkey Creek Rd. Over Central Ga. RR.
WIDENING WIDTH.
TYPE CONSTRUCTION P,S,G
DESIGN COVER 2,7,5
TEST PERFORMED BY. Jim Stiles

CONTRACT B,-,1,-,0,2,2,5,-,8,8
PROJECT. BRZLB-027(1)
COUNTY 7,7 Coweta
DISTRICT 3
AREA ENGINEER. Benjamin E. Rogers

SHEET

CODE	TYPE CONSTRUCTION
PSG	PRESTRESS GIRDER
PTB	POST TENSION BOX
RCG	REINFORCED CONCRETE DECK GIRDER
STG	STEEL GIRDER

SPAN OR POUR NUMBER 1
RATING A
AVERAGE 2.66
STANDARD DEVIATION 0.12
COEFFICIENT OF VARIATION 4.7
NO. OF READINGS 24

INDIVIDUAL MEASUREMENTS

IN.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0								
1								
2				1	4	8	9	2
3								
4								

SPAN OR POUR NUMBER 2
RATING A
AVERAGE 2.72
STANDARD DEVIATION 0.13
COEFFICIENT OF VARIATION 4.9
NO. OF READINGS 32

INDIVIDUAL MEASUREMENTS

IN.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0								
1								
2					5	6	13	7
3	1							
4								

SPAN OR POUR NUMBER 3
RATING B
AVERAGE 2.51
STANDARD DEVIATION 0.16
COEFFICIENT OF VARIATION 6.6
NO. OF READINGS 20

INDIVIDUAL MEASUREMENTS

IN.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0								
1								
2			3	3	8	2	4	
3								
4								

SPAN OR POUR NUMBER
RATING
AVERAGE
STANDARD DEVIATION
COEFFICIENT OF VARIATION
NO. OF READINGS

INDIVIDUAL MEASUREMENTS

IN.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0								
1								
2								
3								
4								

SPAN OR POUR NUMBER
RATING
AVERAGE
STANDARD DEVIATION
COEFFICIENT OF VARIATION
NO. OF READINGS

INDIVIDUAL MEASUREMENTS

IN.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0								
1								
2								
3								
4								

SPAN OR POUR NUMBER
RATING
AVERAGE
STANDARD DEVIATION
COEFFICIENT OF VARIATION
NO. OF READINGS

INDIVIDUAL MEASUREMENTS

IN.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0								
1								
2								
3								
4								

Turkey Creek Rd over RR central GA. BR2LB-077(1) Highland coun Connector

design cover 2 3/4

NW

Span				
#1	2 3/4 1	2 1/2 1	2 3/8 1	2 5/8 6
	2 5/8 1	2 5/8 2	2 1/2 3	2 1/2 4
	2 5/8 6	2 3/4 4	2 5/8 4	2 5/8 7
	2 3/4 2	2 5/8 3	2 5/8 5	2 3/4 8
	2 3/4 3	2 1/2 2	2 3/4 6	2 3/4 9
	2 7/8 1	2 3/4 5	2 3/4 7	2 3/4 2
#2	2 3/4 1	2 3/4 4	2 3/4 8	2 3/4 10
	2 7/8 1	2 7/8 5	2 1/2 2	2 7/8 7
	2 3/4 2	2 3/4 5	2 1/2 3	2 3/4 11
	2 3/4 1	2 3/4 6	2 5/8 2	2 5/8 5
	2 7/8 2	2 5/8 1	2 1/2 4	2 3/4 12
	3 1	2 3/4 7	2 3/4 9	2 5/8 6
	2 7/8 3	2 7/8 6	2 5/8 3	2 3/4 13
	2 7/8 4	2 1/2 1	2 5/8 4	2 1/2 5
Span #3	2 3/4 1	2 3/4 4	2 1/2 4	2 1/2 6
	2 3/4 2	2 5/8 2	2 7/8 1	2 3/4 2
	2 1/2 1	2 1/2 2	2 1/4 2	2 3/8 2
	2 5/8 1	2 1/4 1	2 1/4 3	2 1/2 7
	2 3/4 3	2 1/2 3	2 1/2 5	2 1/2 8

LT. S.R.- 121 RT.

	64	68	66	65
SPAN-1	65	70	75	70
	64	68	70	70
	60	70	67	70
	63	62	64	62
SPAN-2	60	68	69	70
	60	68	66	69
	62	65	62	67
	60	61	60	60
SPAN-3	60	59	60	61
	60	58	60	63
	60	62	63	67
	64	63	62	68
SPAN-4	65	66	66	66
	64	66	66	67
	60	60	60	62
	60	60	60	60
SPAN-5	60	62	66	67
	70	64	63	63
	68	68	62	69

5/2007

INTERDEPARTMENT CORRESPONDENCE

FILE EDS-565(9)01 Burke-Richmond
B11541-03-M00-0
Bridge No. 1
SR121/McBean Creek

METRIC

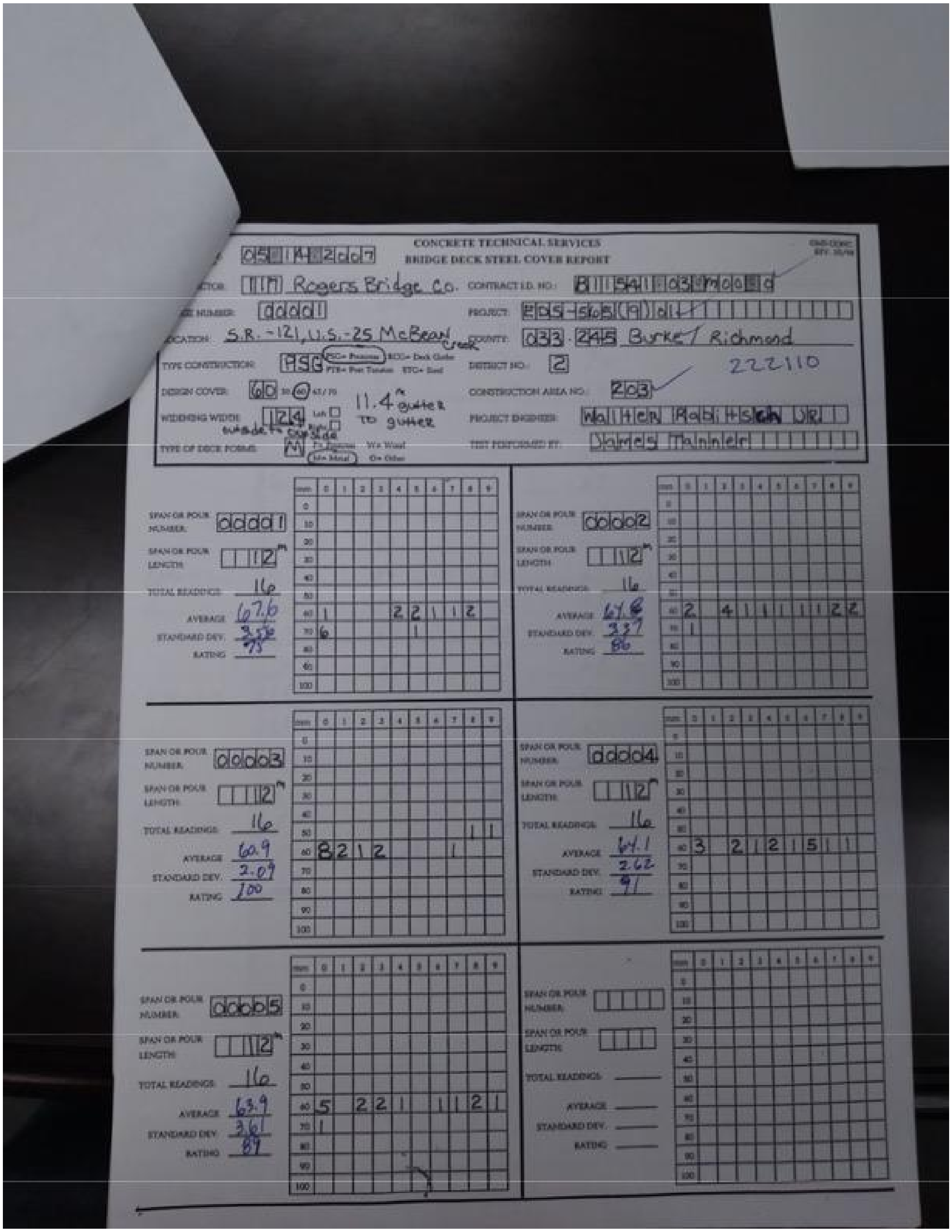
TO Corbett Reynolds, Area Engineer, Louisville

SUBJECT BRIDGE DECK STEEL COVER

BRIDGE NO.	SPAN OR POUR NO.	DESIGN COVER	AVERAGE COVER	RANGE (mm)
1	1	60.	67.6	60. - 75.
	2	60.	64.8	60. - 70.
	3	60.	60.9	58. - 67.
	4	60.	64.1	60. - 68.
	5	60.	63.9	60. - 70.

Based on these results, the rating for span 1 indicates that better control of steel placement is needed. The contractor should take measures to alleviate this problem on future placements.

c: Michael L. Thomas, P.E., District Engineer, Jesup
Rogers Bridge Co.,Inc.



CONCRETE TECHNICAL SERVICES
BRIDGE DECK STEEL COVER REPORT

05 14 2007

CONTRACTOR: 11M Rogers Bridge Co.
CONTRACT I.D. NO.: B 11541 03 M00 E 0
BRIDGE NUMBER: 00001
PROJECT: EDS-565(9) 01
LOCATION: S.R.-121, U.S.-25 McBean Creek
COUNTY: 033 245 Burke / Richmond
TYPE CONSTRUCTION: PSG
DISTRICT NO.: 2
222110
DESIGN COVER: 60
CONSTRUCTION AREA NO.: 203
WIDENING WIDTH: 124
11.4 m gutter to gutter
outside to outside
PROJECT ENGINEER: Walter Rabitsch JR
TYPE OF DECK FORMS: M
TEST PERFORMED BY: James Tanner

SPAN OR POUR NUMBER: 00001
SPAN OR POUR LENGTH: 12 m
TOTAL READINGS: 16
AVERAGE: 67.6
STANDARD DEV.: 3.56
RATING: 75

60: 1 2 2 1 1 2
70: 6 1

SPAN OR POUR NUMBER: 00002
SPAN OR POUR LENGTH: 12 m
TOTAL READINGS: 16
AVERAGE: 64.6
STANDARD DEV.: 3.37
RATING: 86

60: 2 4 1 1 1 1 2 2
70: 1

SPAN OR POUR NUMBER: 00003
SPAN OR POUR LENGTH: 12 m
TOTAL READINGS: 16
AVERAGE: 60.9
STANDARD DEV.: 2.09
RATING: 100

50: 1 1
60: 8 2 1 2 1

SPAN OR POUR NUMBER: 00004
SPAN OR POUR LENGTH: 12 m
TOTAL READINGS: 16
AVERAGE: 64.1
STANDARD DEV.: 2.62
RATING: 91

60: 3 2 1 2 1 5 1 1

SPAN OR POUR NUMBER: 00005
SPAN OR POUR LENGTH: 12 m
TOTAL READINGS: 16
AVERAGE: 63.9
STANDARD DEV.: 3.61
RATING: 89

60: 5 2 2 1 1 1 2 1
70: 1

SPAN OR POUR NUMBER:
SPAN OR POUR LENGTH:
TOTAL READINGS:
AVERAGE:
STANDARD DEV.:
RATING:

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information NHS-0005-00(071)01

Bridge No. 15

Span No.: 13
Type Beams: Prestress
Span Length (ft.): 56'
Design Cover: 2.25

Average = 2.24
Standard Deviation = 0.11
Rating = 100
Total Readings = 12

Low = 2.10
High = 2.40

Beam Bays

Bent #		
13	2.20	2.30
	2.20	2.20
	2.20	2.20
	2.40	2.40
	2.20	2.40
	2.10	2.10

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	2	6	1	3	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE

95 - 100	EXCELLENT
90 - 94	GOOD
80 - 89	FAIR
70 - 79	MARGINAL
BELOW 70	NEEDS IMPROVEMENT

INTERDEPARTMENT CORRESPONDENCE

9/2008

FILE NHS-0005-00(071)01 Glynn
B12515-06-000-0
PI No. 0005071
Bridge No. 15 Right Outside Widening
I-95/Gibson Creek

TO Larry C. Barnes, Area Engineer, I-95 Reconstruction

SUBJECT BRIDGE DECK STEEL COVER

BRIDGE NO.	SPAN OR POUR NO.	DESIGN COVER	AVERAGE COVER	RANGE (IN.)
15ROW	1	2.25	2.22	2.00-2.40
	2	2.25	2.25	1.90-2.40
	3	2.25	2.21	2.00-2.40
	4	2.25	2.15	2.00-2.30
	5	2.25	2.23	2.10-2.40
	6	2.25	2.31	2.10-2.40
	7	2.25	2.26	2.00-2.40
	8	2.25	2.11	1.70-2.50
	9	2.25	2.23	1.80-2.50
	10	2.25	2.37	2.10-2.50
	11	2.25	2.33	2.20-2.40
	12	2.25	2.23	1.90-2.40
	13	2.25	2.24	2.10-2.40

c: Glenn Durrence, P.E., District Engineer, Jesup
Skanska USA Civil Southeast, Inc.

CONCRETE TECHNICAL SERVICES
BRIDGE DECK STEEL COVER REPORT

Project Information

Report Date:	9/12/2008
Contract ID No.:	B12515-06-000-0
P.I. No.:	0005071
Project No.:	NHS-0005-00(071)01
County No.:	Glynn -
District No.:	5
Construction Area No.:	503-Brunswick
Project Engineer:	Jeffery Moody
Contractor:	48 - Skanska
Bridge No.:	15 Right Stage 1
Location:	I-95 NB over Gibson Creek
Type Deck Forms:	Metal
Widening Location:	Outside
Widening Width (ft.):	22' 5 7/16"
Design Cover (in.):	2.25
Deck Width (ft.):	
Deck Thickness (in.):	
Number of Spans:	13
Test Performed By:	Jason Waters

Remarks: Skanska is the bridge contractor. A core was taken to verify the steel cover.

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information NHS-0005-00(071)01

Bridge No. 15

Span No.: 1
Type Beams: Prestress
Span Length (ft.): 56'
Design Cover: 2.25

Average = 2.22
Standard Deviation = 0.11
Rating = 99
Total Readings = 12

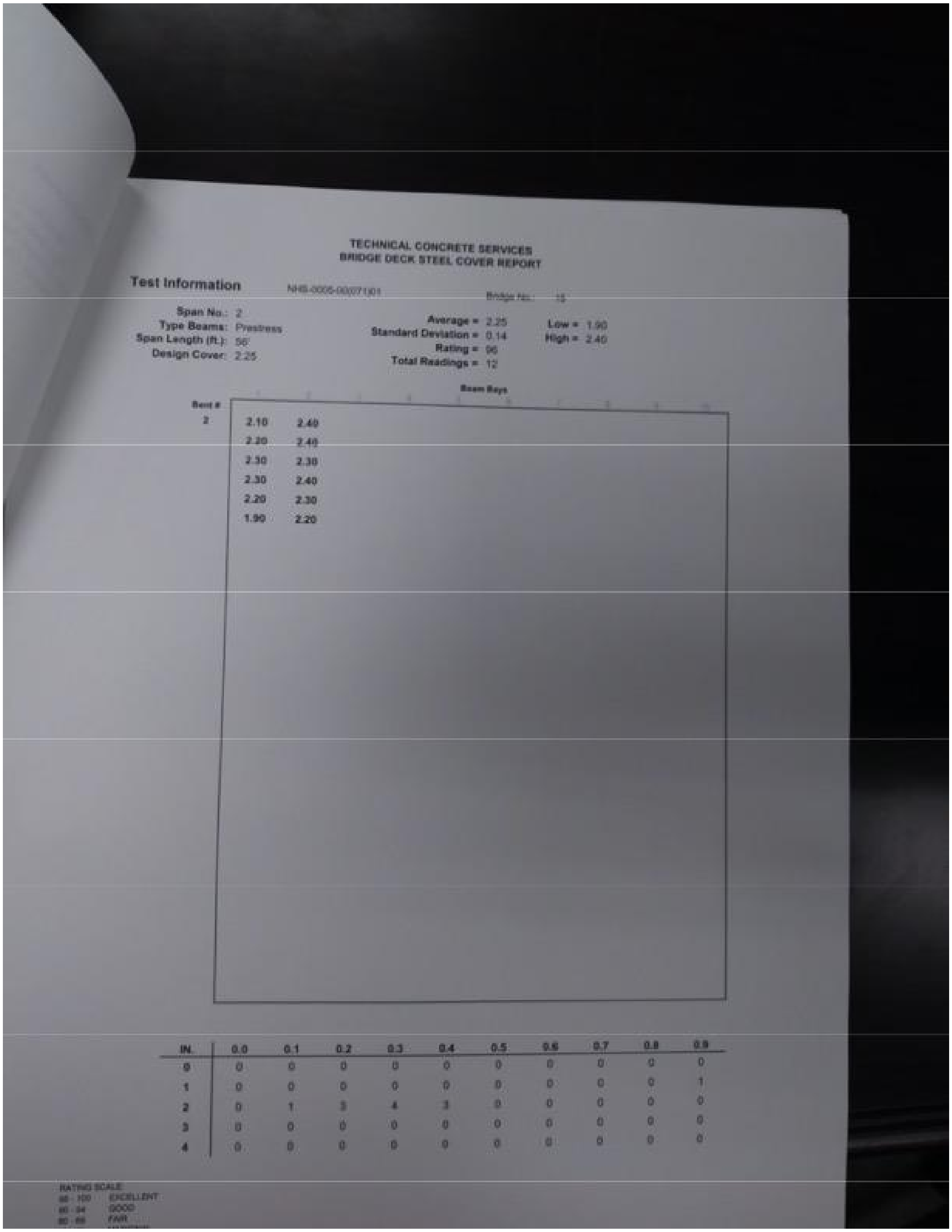
Low = 2.00
High = 2.40

Beam Bays

Bent #		
1	2.00	2.30
	2.20	2.30
	2.20	2.30
	2.40	2.20
	2.20	2.30
	2.10	2.10

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	1	2	4	4	1	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
96 - 100 EXCELLENT



TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information NHS-0005-00071(01) Bridge No.: 15

Span No.: 2
Type Beams: Prestress
Span Length (ft.): 56'
Design Cover: 2.25

Average = 2.25
Standard Deviation = 0.14
Rating = 96
Total Readings = 12

Low = 1.90
High = 2.40

Beam Bays

Bent #	1	2
2	2.10	2.40
	2.20	2.40
	2.30	2.30
	2.30	2.40
	2.20	2.30
	1.90	2.20

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	1
2	0	1	3	4	3	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
[illegible] EXCELLENT
[illegible] GOOD
[illegible] FAIR

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information

NHS-0005-00(07)301   Bridge No.: 15

Span No.: 3
Type Beams: Prestress
Span Length (ft.): 56'
Design Cover: 2.25

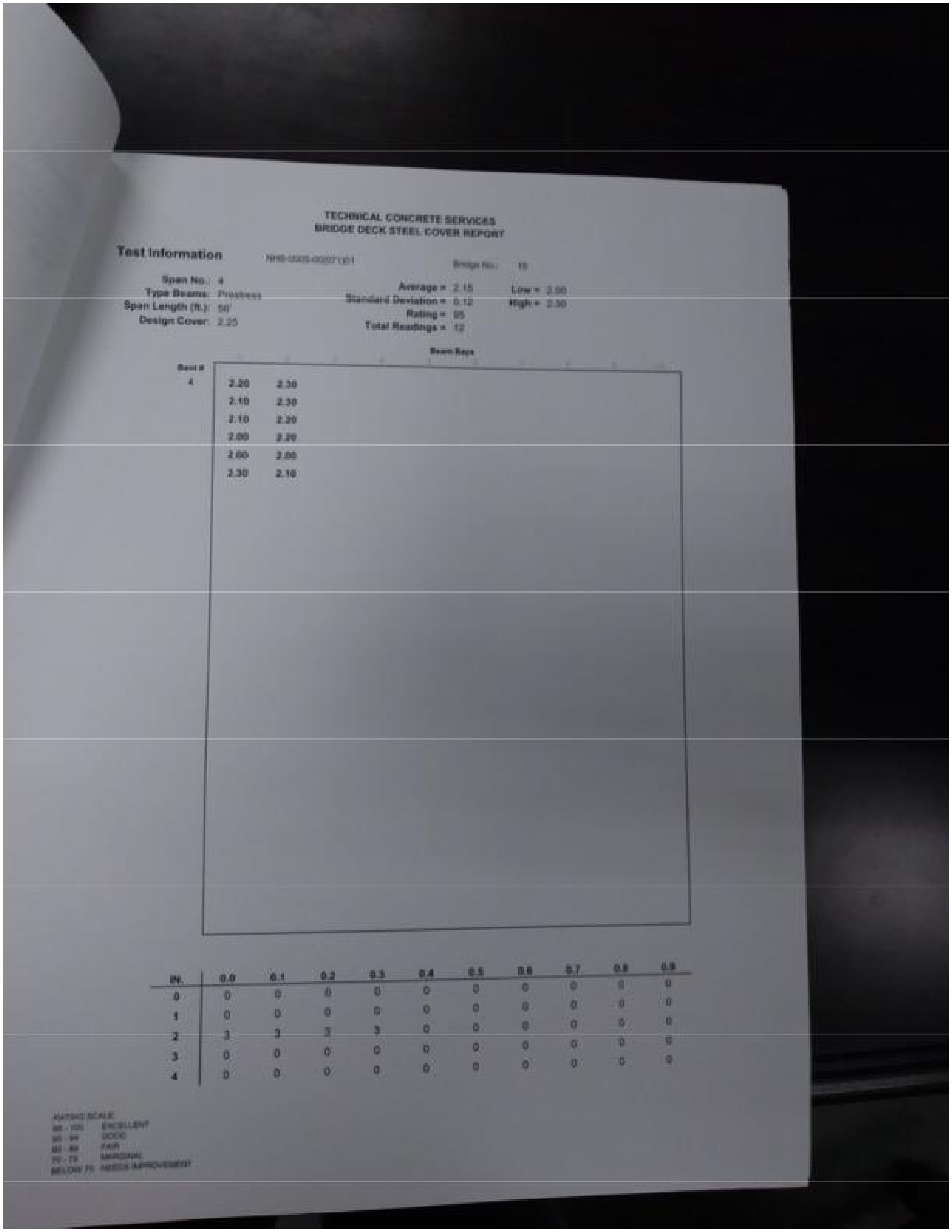
Average = 2.21   Low = 2.00
Standard Deviation = 0.16   High = 2.40
Rating = 94
Total Readings = 12

Beam Bays

Bent #		
3	2.00	2.20
	2.10	2.30
	2.10	2.30
	2.00	2.40
	2.00	2.40
	2.30	2.40

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	3	2	1	3	3	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
[illegible] - 100 EXCELLENT
[illegible] - 94 GOOD
[illegible] - [illegible] FAIR
70 - 79 MARGINAL
BELOW 70 NEEDS IMPROVEMENT



TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information NHB-0005-00(071)01 Bridge No.: 19

Span No.: 4
Type Beams: Prestress
Span Length (ft.): 50'
Design Cover: 2.25

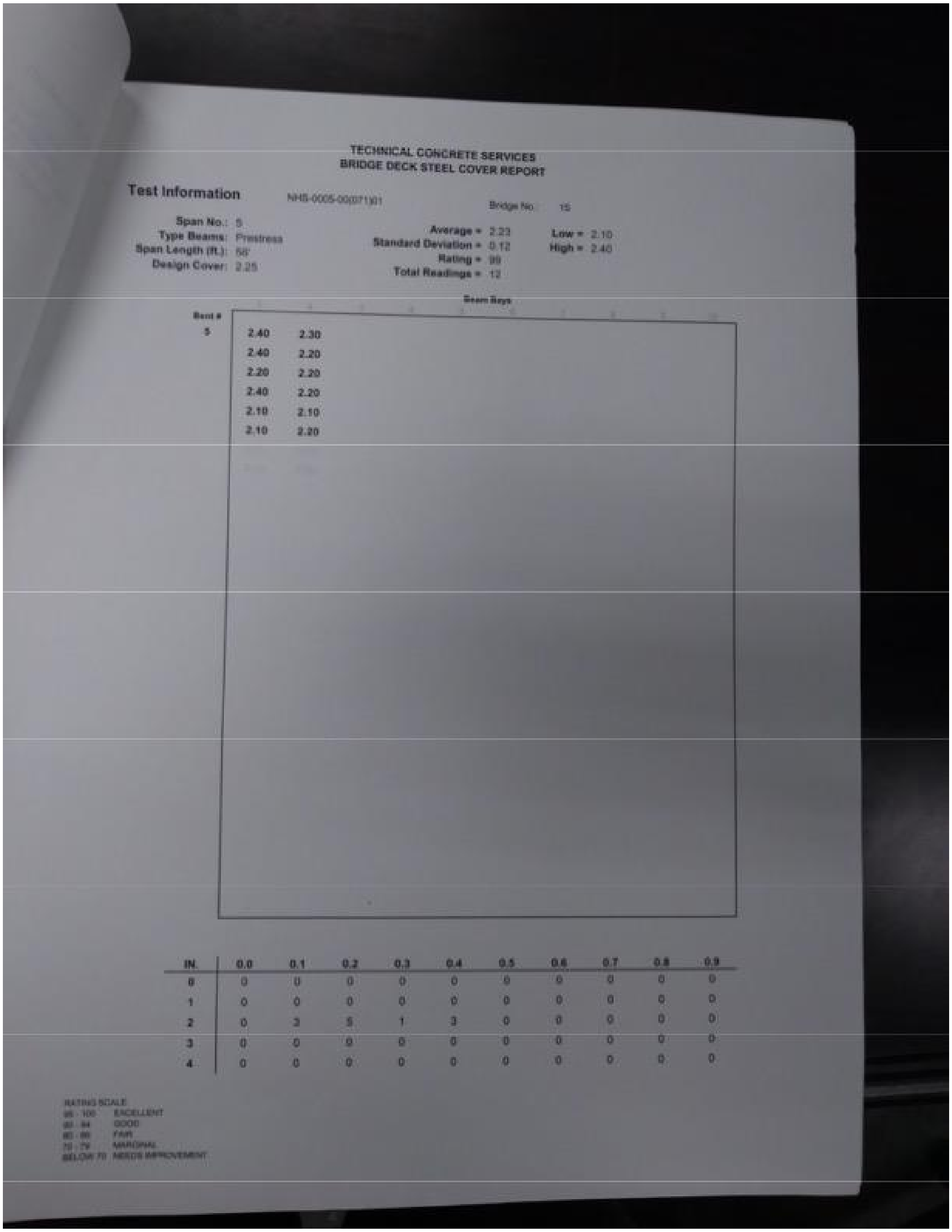
Average = 2.15 Low = 2.00
Standard Deviation = 0.12 High = 2.30
Rating = 95
Total Readings = 12

Beam Bays

Bent #		
4	2.20	2.30
	2.10	2.30
	2.10	2.20
	2.00	2.20
	2.00	2.00
	2.30	2.10

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	3	3	3	3	0	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
[illegible] EXCELLENT
[illegible] GOOD
[illegible] FAIR
[illegible] MARGINAL
BELOW [illegible] NEEDS IMPROVEMENT



TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information NHS-0005-00(071)01 Bridge No. 15

Span No.: 5
Type Beams: Prestress
Span Length (ft.): 56'
Design Cover: 2.25

Average = 2.23
Standard Deviation = 0.12
Rating = 99
Total Readings = 12

Low = 2.10
High = 2.40

Beam Bays

Bent #		
5	2.40	2.30
	2.40	2.20
	2.20	2.20
	2.40	2.20
	2.10	2.10
	2.10	2.20

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	3	5	1	3	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
EXCELLENT
GOOD
FAIR
MARGINAL
BELOW 70 NEEDS IMPROVEMENT

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information NHS-0005-00(071)01 Bridge No.: 18

Span No.: 6
Type Beams: Prestress
Span Length (ft.): 80'
Design Cover: 2.25

Average = 2.31
Standard Deviation = 0.11
Rating = 98
Total Readings = 16

Low = 2.10
High = 2.40

Beam Bays

Bent #		
6	2.40	2.10
	2.40	2.40
	2.30	2.10
	2.40	2.20
	2.20	2.30
	2.40	2.40
	2.40	2.40
	2.20	2.30

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	2	3	3	8	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
[illegible] - 100 EXCELLENT
[illegible] - 94 GOOD
[illegible] FAIR
70 - 79 MARGINAL
BELOW 70 NEEDS IMPROVEMENT

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information

NHS-0005-00(071)01

Bridge No.: 15

Span No.: 7
Type Beams: Prestress
Span Length (ft.): 80'
Design Cover: 2.25

Average = 2.26
Standard Deviation = 0.14
Rating = 97
Total Readings = 16

Low = 2.00
High = 2.40

Beam Bays

Bent #		
7	2.30	2.20
	2.40	2.40
	2.30	2.40
	2.20	2.10
	2.40	2.20
	2.40	2.10
	2.40	2.00
	2.20	2.10

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	1	3	4	2	6	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE:
95 - 100 EXCELLENT
90 - 94 GOOD
80 - 89 FAIR
70 - 79 MARGINAL
BELOW 70 NEEDS IMPROVEMENT

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information

NHS-0005-00(071)01

Bridge No.: 15

Span No.:	8	**Average =**	2.11	**Low =**	1.70
Type Beams:	Prestress	**Standard Deviation =**	0.17	**High =**	2.50
Span Length (ft.):	80'	**Rating =**	87		
Design Cover:	2.25	**Total Readings =**	16		

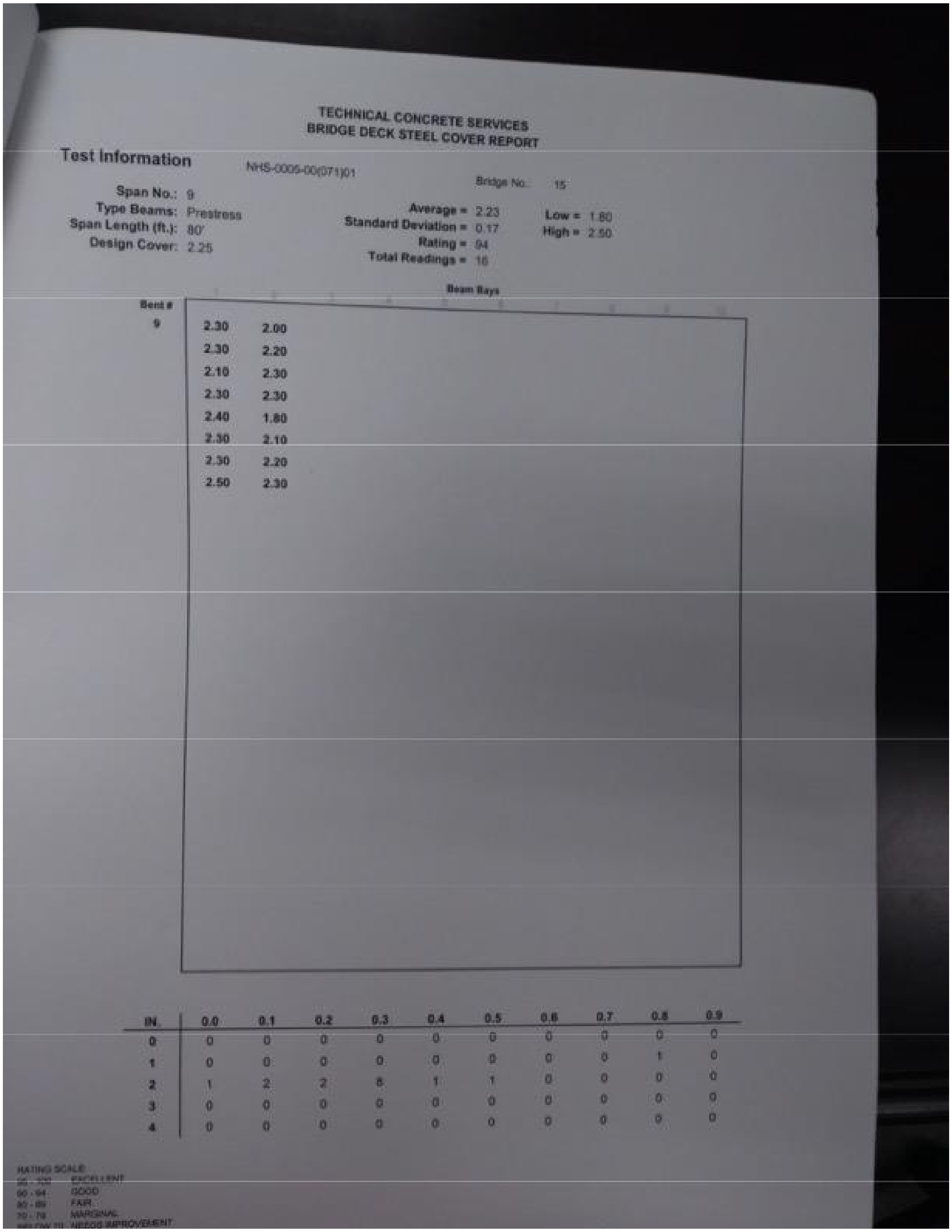
Beam Bays

Bent #		
8	2.10	2.20
	2.10	2.00
	2.00	2.00
	2.30	2.20
	2.20	2.20
	2.20	1.70
	2.10	2.00
	2.50	2.00

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	1	0	0
2	5	3	5	1	0	1	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE

95 - 100	EXCELLENT
90 - 94	GOOD
80 - 89	FAIR
70 - 79	MARGINAL
BELOW 70	NEEDS IMPROVEMENT



TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information

NHS-0005-00(071)01

Bridge No.: 15

Span No.: 9
Type Beams: Prestress
Span Length (ft.): 80'
Design Cover: 2.25

Average = 2.23
Standard Deviation = 0.17
Rating = 94
Total Readings = 16

Low = 1.80
High = 2.50

Beam Bays

Bent #		
9	2.30	2.00
	2.30	2.20
	2.10	2.30
	2.30	2.30
	2.40	1.80
	2.30	2.10
	2.30	2.20
	2.50	2.30

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	1	0
2	1	2	2	8	1	1	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
95 - 100 EXCELLENT
90 - 94 GOOD
80 - 89 FAIR
70 - 79 MARGINAL
BELOW 70 NEEDS IMPROVEMENT

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information

NHS-0005-00(071)01

Bridge No.: 15

Span No.: 10
Type Beams: Prestress
Span Length (ft.): 56'
Design Cover: 2.25

Average = 2.37
Standard Deviation = 0.11
Rating = 95
Total Readings = 12

Low = 2.10
High = 2.50

Beam Bays

Band #		
10	2.40	2.40
	2.40	2.40
	2.40	2.40
	2.40	2.50
	2.40	2.20
	2.40	2.10

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	1	1	0	9	1	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
95 - 100 EXCELLENT
90 - 94 GOOD
80 - 89 FAIR
70 - 79 MARGINAL
BELOW 70 NEEDS IMPROVEMENT

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information

NHS-0005-00(071)01

Bridge No.: 15

Span No.: 11
Type Beams: Prestress
Span Length (ft.): 56'
Design Cover: 2.25

Average = 2.33
Standard Deviation = 0.08
Rating = 100
Total Readings = 12

Low = 2.20
High = 2.40

Beam Bays

Bent #		
11	2.30	2.40
	2.40	2.20
	2.40	2.40
	2.40	2.30
	2.40	2.30
	2.30	2.20

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	0	2	4	6	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
95 - 100 EXCELLENT
90 - 94 GOOD
80 - 89 FAIR
70 - 79 MARGINAL
BELOW 70 NEEDS IMPROVEMENT

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information

NHS-0005-00(071)01

Bridge No.: 15

Span No.: 12
Type Beams: Prestress
Span Length (ft.): 56'
Design Cover: 2.25

Average = 2.23
Standard Deviation = 0.15
Rating = 96
Total Readings = 12

Low = 1.90
High = 2.40

Beam Bays

Bent #		
12	1.90	2.30
	2.10	2.40
	2.20	2.40
	2.20	2.30
	2.10	2.20
	2.40	2.30

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	1
2	0	2	3	3	3	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE:
[illegible] - 100 EXCELLENT
[illegible] GOOD
[illegible] FAIR
[illegible] MARGINAL
BELOW 70 NEEDS IMPROVEMENT

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information CSSTP-0307-00(217) Bridge No.: 2

Span No.: 3
Type Beams: Prestress
Span Length (ft.): 57
Design Cover: 2.75

Average = 2.70
Standard Deviation = 0.05
Rating = 100
Total Readings = 30

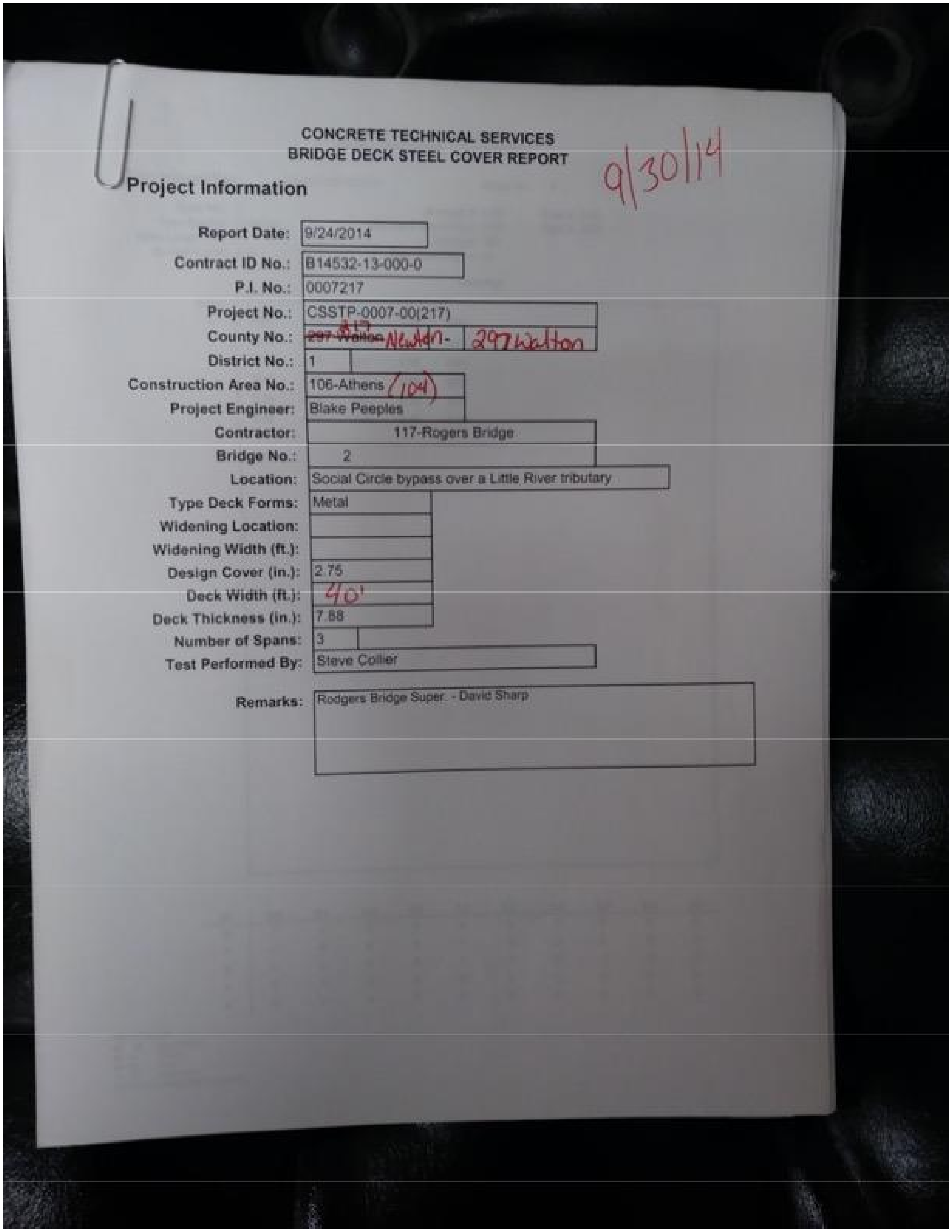
Low = 2.60
High = 2.80

Beam Bays

Bent #					
3	2.70	2.60	2.70	2.70	2.70
	2.70	2.70	2.70	2.70	2.60
	2.70	2.70	2.70	2.70	2.70
	2.70	2.70	2.70	2.70	2.70
	2.80	2.70	2.70	2.70	2.60
	2.80	2.80	2.80	2.70	2.60

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	4	22	4	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
95 - 100 EXCELLENT
85 - 94 GOOD
80 - 84 FAIR
70 - 79 MARGINAL
BELOW 70 NEEDS IMPROVEMENT



CONCRETE TECHNICAL SERVICES
BRIDGE DECK STEEL COVER REPORT

9/30/14

Project Information

Report Date:	9/24/2014
Contract ID No.:	B14532-13-000-0
P.I. No.:	0007217
Project No.:	CSSTP-0007-00(217)
County No.:	~~297 Walton~~ 217 Newton- | 297 Walton
District No.:	1
Construction Area No.:	106-Athens (104)
Project Engineer:	Blake Peeples
Contractor:	117-Rogers Bridge
Bridge No.:	2
Location:	Social Circle bypass over a Little River tributary
Type Deck Forms:	Metal
Widening Location:	
Widening Width (ft.):	
Design Cover (in.):	2.75
Deck Width (ft.):	40'
Deck Thickness (in.):	7.88
Number of Spans:	3
Test Performed By:	Steve Collier
Remarks:	Rodgers Bridge Super. - David Sharp

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information CBSTP-0007-00(217) Bridge No.: 3

Span No.:	1	Average =	2.66	Low =	2.50
Type Beams:	Prestress	Standard Deviation =	0.07	High =	2.80
Span Length (ft.):	67	Rating =	100		
Design Cover:	2.75	Total Readings =	30		

Beam Bays

Band #					
1	2.60	2.70	2.70	2.70	2.80
	2.70	2.70	2.70	2.60	2.60
	2.70	2.60	2.60	2.60	2.60
	2.70	2.60	2.70	2.60	2.50
	2.70	2.70	2.60	2.70	2.60
	2.60	2.60	2.70	2.60	2.60

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	1	13	13	3	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
[illegible] - 100 EXCELLENT
[illegible] GOOD
[illegible] FAIR
[illegible] MARGINAL
BELOW [illegible] NEEDS IMPROVEMENT

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information CSSTP-0007-00(217) Bridge No. 2

Span No.: 2
Type Beams: Prestress
Span Length (Ft.): 83
Design Cover: 2.75

Average = 2.68
Standard Deviation = 0.09
Rating = 100
Total Readings = 45

Low = 2.50
High = 2.80

Beam Bays

Bent #					
2	2.80	2.50	2.70	2.70	2.60
	2.70	2.60	2.80	2.60	2.70
	2.80	2.70	2.70	2.70	2.80
	2.70	2.80	2.70	2.60	2.70
	2.60	2.70	2.70	2.70	2.50
	2.60	2.60	2.70	2.80	2.60
	2.80	2.60	2.70	2.70	2.80
	2.80	2.70	2.70	2.60	2.60
	2.80	2.80	2.70	2.50	2.60

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	3	13	19	10	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
[illegible] EXCELLENT
[illegible] GOOD
[illegible] FAIR
[illegible] MARGINAL
BELOW [illegible] NEEDS IMPROVEMENT

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information EDS00-0072-00(028) Bridge No. 1

Span No.: 3
Type Beams: Prestress
Span Length (ft.): 75
Design Cover: 2.75

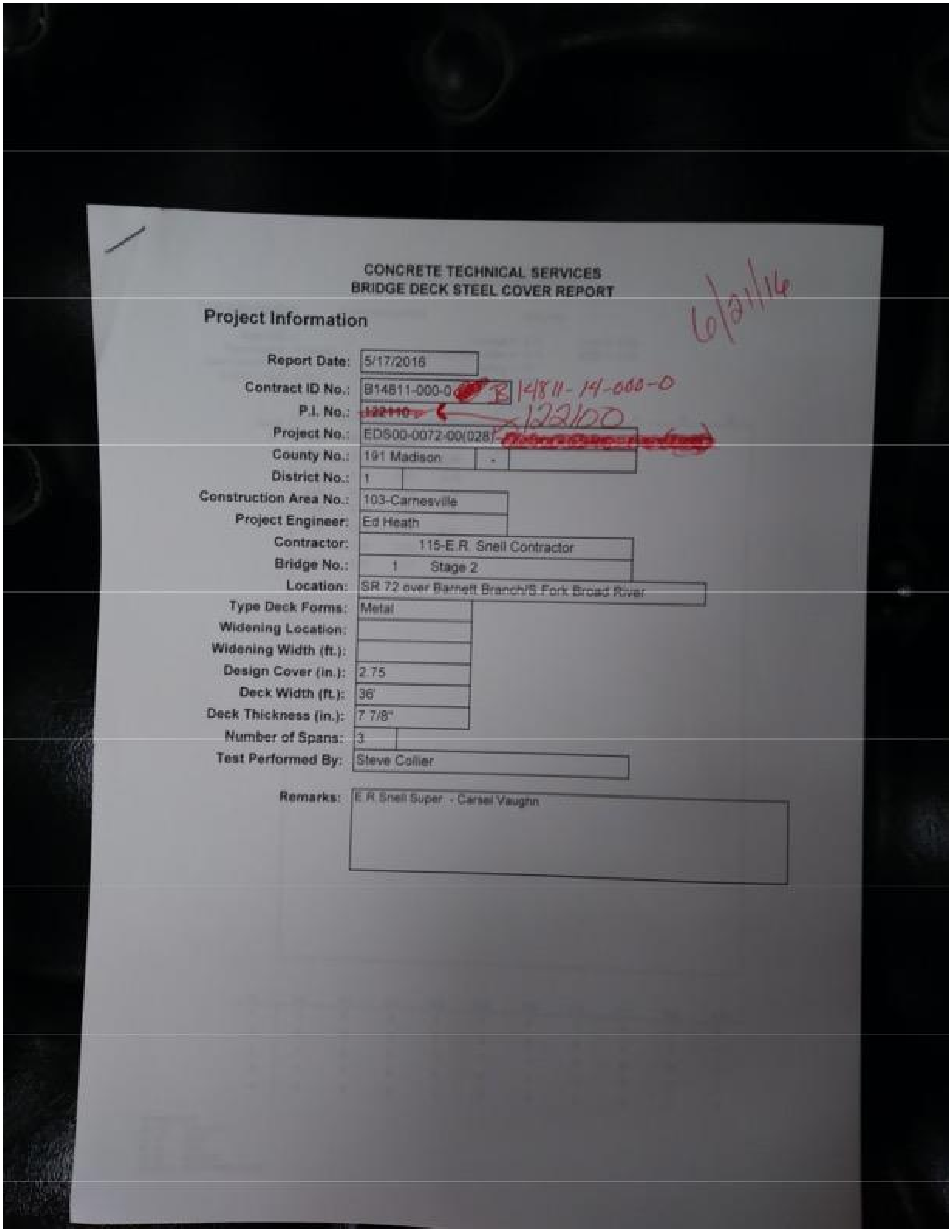
Average = 2.66
Standard Deviation = 0.08
Rating = 98
Total Readings = 32

Low = 2.50
High = 2.90

Bent #	Beam Bays 1	2	3	4	5	6	7	8	9	10
3	2.60	2.80	2.60	2.70						
	2.70	2.60	2.60	2.70						
	2.60	2.50	2.60	2.60						
	2.60	2.60	2.60	2.90						
	2.70	2.70	2.70	2.80						
	2.60	2.60	2.70	2.80						
	2.70	2.60	2.60	2.70						
	2.60	2.60	2.60	2.70						

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	1	17	10	3	1
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
95 - 100 EXCELLENT
90 - 94 GOOD
80 - 89 FAIR
70 - 79 MARGINAL
BELOW 70 NEEDS IMPROVEMENT

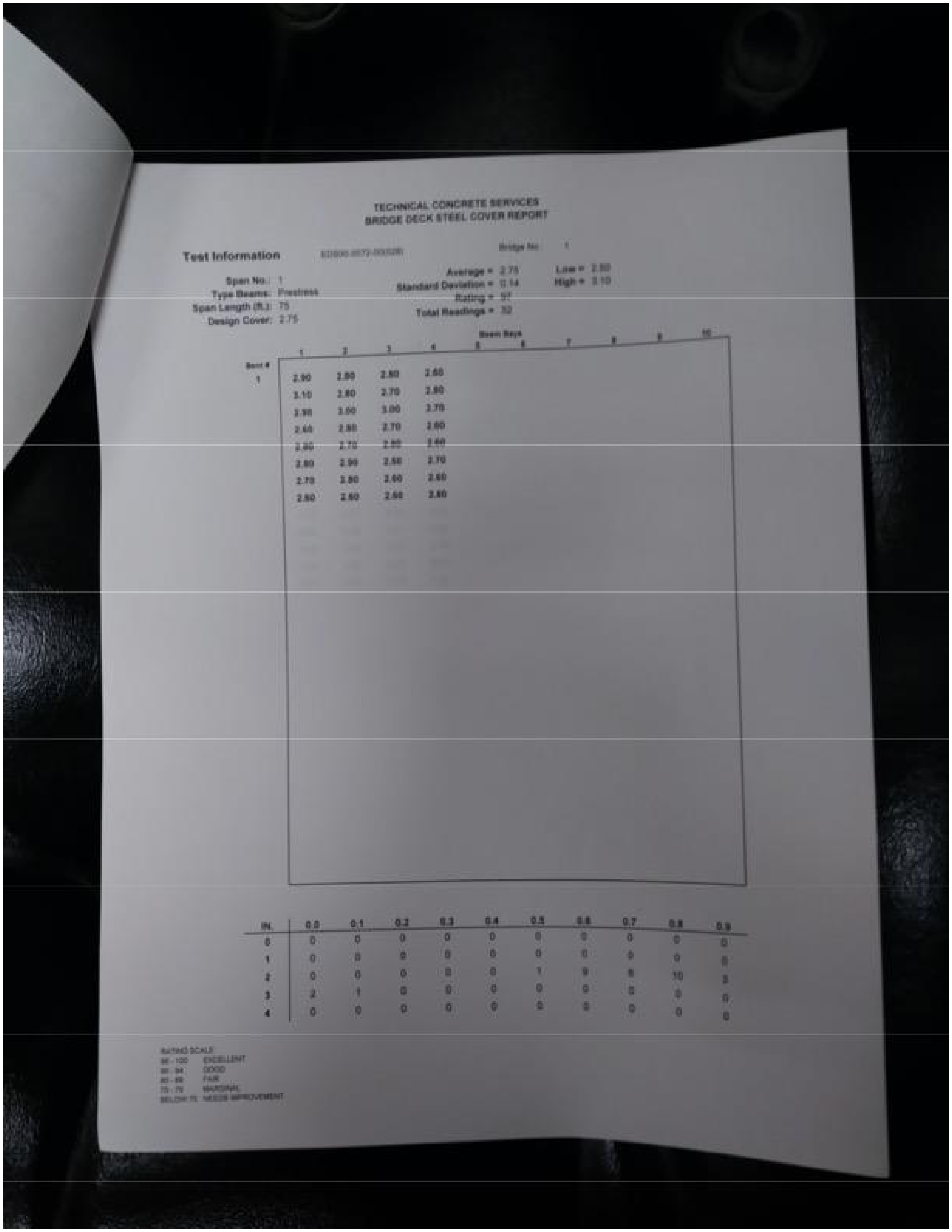


CONCRETE TECHNICAL SERVICES
BRIDGE DECK STEEL COVER REPORT

6/21/16

Project Information

Report Date:	5/17/2016
Contract ID No.:	B14811-000-0 B14811-14-000-0
P.I. No.:	~~122110~~ 122100
Project No.:	EDS00-0072-00(028)
County No.:	191 Madison -
District No.:	1
Construction Area No.:	103-Carnesville
Project Engineer:	Ed Heath
Contractor:	115-E.R. Snell Contractor
Bridge No.:	1 Stage 2
Location:	SR 72 over Barnett Branch/S Fork Broad River
Type Deck Forms:	Metal
Widening Location:	
Widening Width (ft.):	
Design Cover (in.):	2.75
Deck Width (ft.):	36'
Deck Thickness (in.):	7 7/8"
Number of Spans:	3
Test Performed By:	Steve Collier
Remarks:	E.R Snell Super - Carsel Vaughn



TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information [illegible] Bridge No: 1

Span No.: 1
Type Beams: Prestress
Span Length (ft.): 75
Design Cover: 2.75

Average = 2.75
Standard Deviation = 0.14
Rating = 97
Total Readings = 32

Low = 2.50
High = 3.10

Beam Bays

Bent #	1	2	3	4	5	6	7	8	9	10
1	2.90	2.80	2.80	2.60						
	3.10	2.80	2.70	2.80						
	2.90	3.00	3.00	2.70						
	2.60	2.80	2.70	2.80						
	2.80	2.70	2.80	2.60						
	2.80	2.90	2.50	2.70						
	2.70	2.80	2.60	2.60						
	2.80	2.60	2.60	2.80						

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	1	9	8	10	3
3	2	1	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
[illegible] EXCELLENT
[illegible] GOOD
[illegible] FAIR
75 - 79 MARGINAL
BELOW 75 NEEDS IMPROVEMENT

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information

ED500-0072-00(028) Bridge No. 1

Span No.: 2
Type Beams: Prestress
Span Length (ft.): 125
Design Cover: 2.75

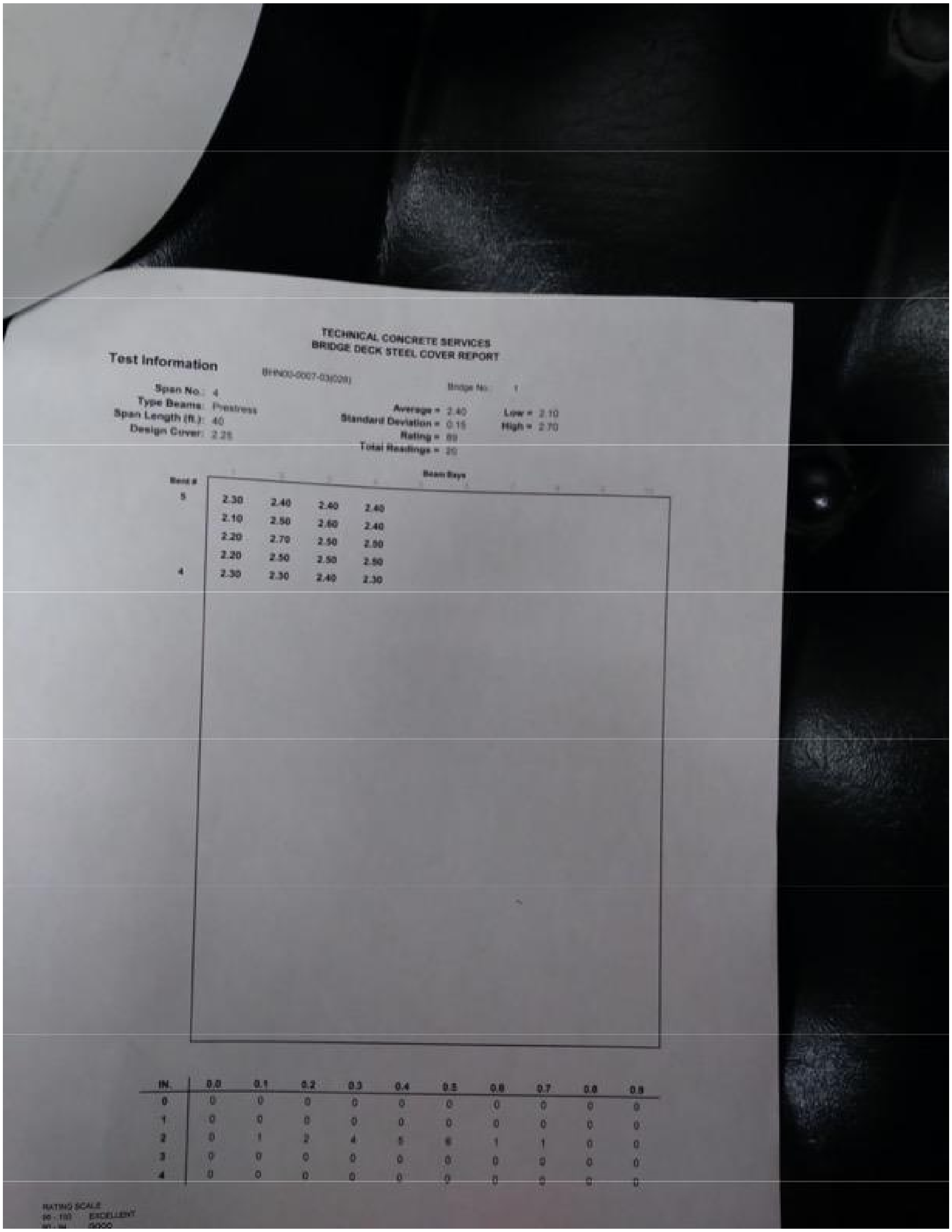
Average = 2.71
Standard Deviation = 0.11
Rating = 99
Total Readings = 52

Low = 2.50
High = 3.00

Bent #	Beam Bays 1	2	3	4	5	6	7	8	9	10
2	2.80	2.70	2.60	2.70						
	2.70	2.60	2.60	2.70						
	2.60	2.70	2.60	2.80						
	2.70	2.70	2.80	2.60						
	2.60	2.80	2.90	2.80						
	2.70	2.70	2.80	2.60						
	2.60	2.80	2.90	2.80						
	2.60	2.80	2.60	2.90						
	2.70	2.90	2.60	2.70						
	2.80	2.80	2.70	2.60						
	2.90	2.60	2.60	2.70						
	3.00	2.80	2.70	2.60						
	2.80	2.50	2.70	2.60						

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	1	17	15	13	5
3	1	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE:
95 - 100 EXCELLENT
90 - 94 GOOD
80 - 89 FAIR
70 - 79 MARGINAL
BELOW 70 NEEDS IMPROVEMENT



TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information

BHN00-0007-03(028)

Bridge No.: 1

Span No.: 4
Type Beams: Prestress
Span Length (ft.): 40
Design Cover: 2.25

Average = 2.40
Standard Deviation = 0.15
Rating = 89
Total Readings = 20

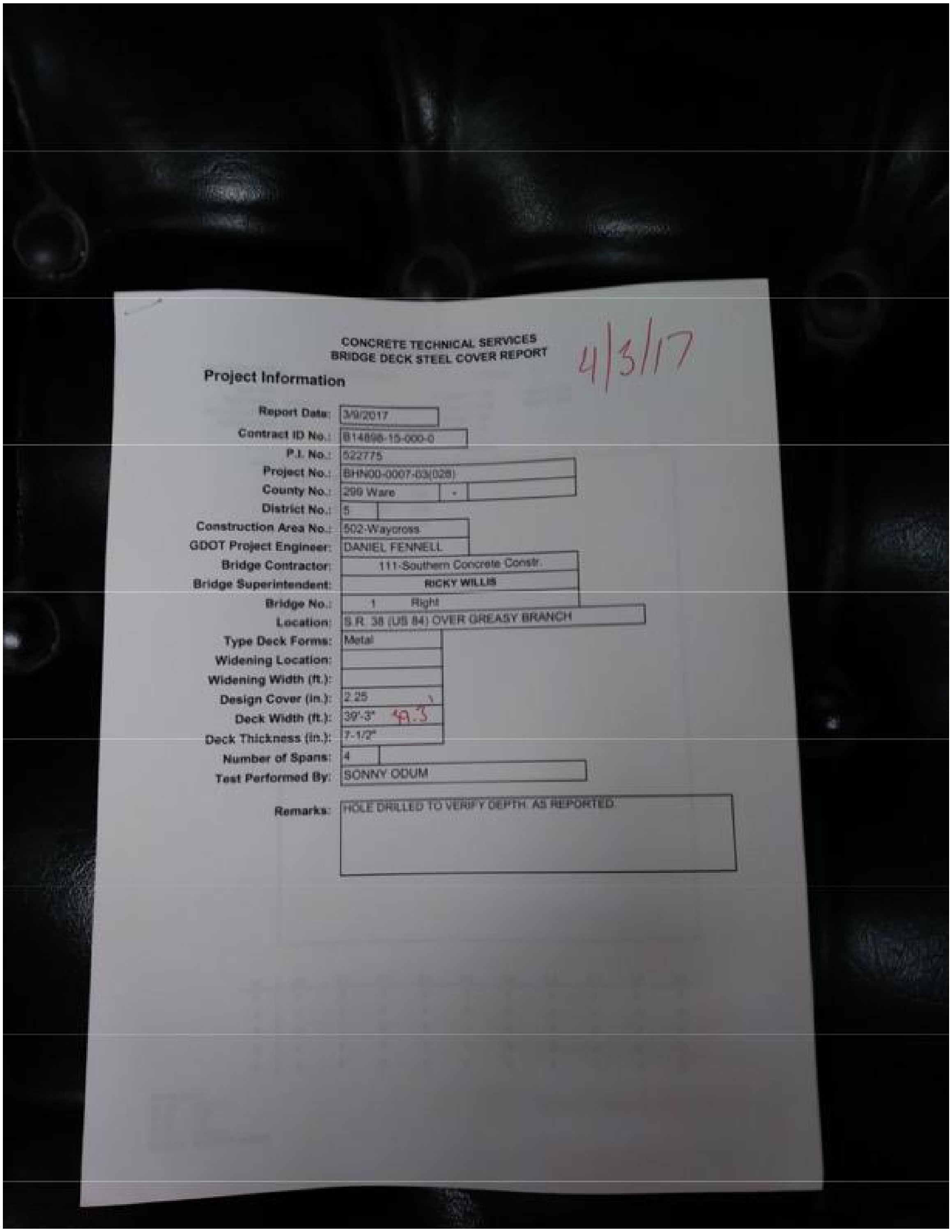
Low = 2.10
High = 2.70

Beam Bays

Bent #				
5	2.30	2.40	2.40	2.40
	2.10	2.50	2.60	2.40
	2.20	2.70	2.50	2.50
	2.20	2.50	2.50	2.50
4	2.30	2.30	2.40	2.30

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	1	2	4	5	6	1	1	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
90 - 100 EXCELLENT
80 - 89 GOOD

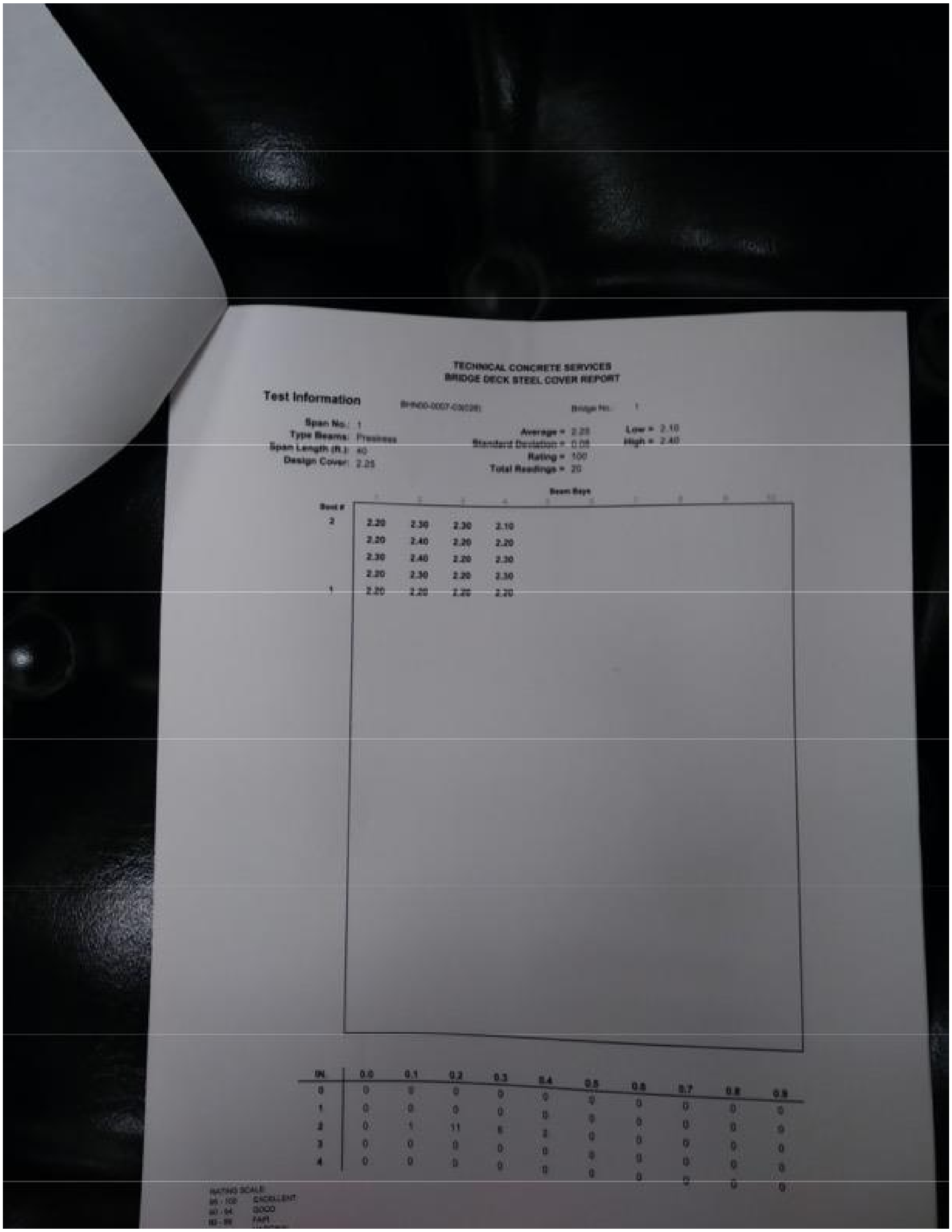


CONCRETE TECHNICAL SERVICES
BRIDGE DECK STEEL COVER REPORT

4/3/17

Project Information

Report Date:	3/9/2017
Contract ID No.:	B14898-15-000-0
P.I. No.:	522775
Project No.:	BHN00-0007-03(028)
County No.:	299 Ware
District No.:	5
Construction Area No.:	502-Waycross
GDOT Project Engineer:	DANIEL FENNELL
Bridge Contractor:	111-Southern Concrete Constr.
Bridge Superintendent:	RICKY WILLIS
Bridge No.:	1 Right
Location:	S.R. 38 (US 84) OVER GREASY BRANCH
Type Deck Forms:	Metal
Widening Location:	
Widening Width (ft.):	
Design Cover (in.):	2.25
Deck Width (ft.):	39'-3" 39.3
Deck Thickness (in.):	7-1/2"
Number of Spans:	4
Test Performed By:	SONNY ODUM
Remarks:	HOLE DRILLED TO VERIFY DEPTH. AS REPORTED.


TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information

Bridge No.: 1

Span No.: 1
Type Beams: Prestress
Span Length (ft.): 40
Design Cover: 2.25

Average = 2.25
Standard Deviation = 0.08
Rating = 100
Total Readings = 20

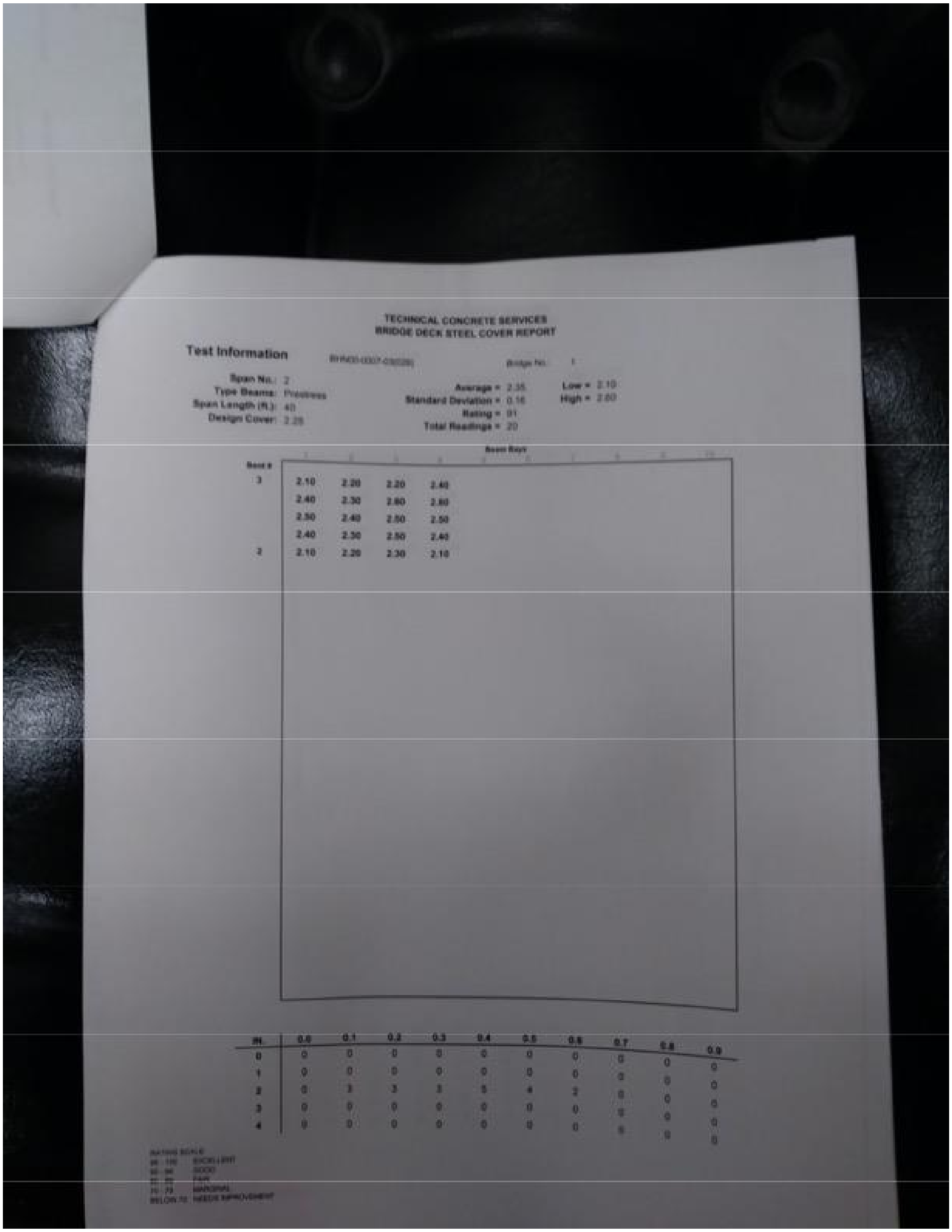
Low = 2.10
High = 2.40

Beam Bays

Bent #	1	2	3	4
2	2.20	2.30	2.30	2.10
	2.20	2.40	2.20	2.20
	2.30	2.40	2.20	2.30
	2.20	2.30	2.20	2.30
1	2.20	2.20	2.20	2.20

IN	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	1	11	6	2	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
EXCELLENT
GOOD
FAIR



TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information [illegible] Bridge No. 1

Span No.: 2
Type Beams: Prestress
Span Length (ft.): 40
Design Cover: 2.25

Average = 2.35
Standard Deviation = 0.16
Rating = 91
Total Readings = 20

Low = 2.10
High = 2.60

Beam Rays

Bent #	1	2	3	4	5	6	7	8	9	10
3	2.10	2.20	2.20	2.40						
	2.40	2.30	2.60	2.60						
	2.50	2.40	2.50	2.50						
	2.40	2.30	2.50	2.40						
2	2.10	2.20	2.30	2.10						

in.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	3	3	3	5	4	2	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
90 - 100 EXCELLENT
[illegible] GOOD
[illegible] FAIR
[illegible] MARGINAL
BELOW 70 NEEDS IMPROVEMENT

DEPARTMENT OF TRANSPORTATION
STATE OF GEORGIA

INTERDEPARTMENT CORRESPONDENCE

FILE **CSBRG-0007-00(030) DAWSON**
B14938-15-000-1
Bridge No. 1 Stage # 1
SR 9 / Etowah River
PI 0007030

OFFICE Materials and Testing
Forest Park, Georgia

DATE July 5, 2018

FROM Monica L. Flournoy, P.E., State Materials Engineer

TO Brent Cook, P.E., District Engineer, Gainesville

SUBJECT **BRIDGE DECK STEEL COVER**

Pachometer measurements to determine the concrete cover over top reinforcing steel on the referenced project have been obtained.

Listed below is a summary of these measurements:

BRIDGE NO.	SPAN OR POUR NO.	DESIGN COVER	AVERAGE COVER	RANGE (in.)
1	1	2.75	2.78	2.50 – 3.00

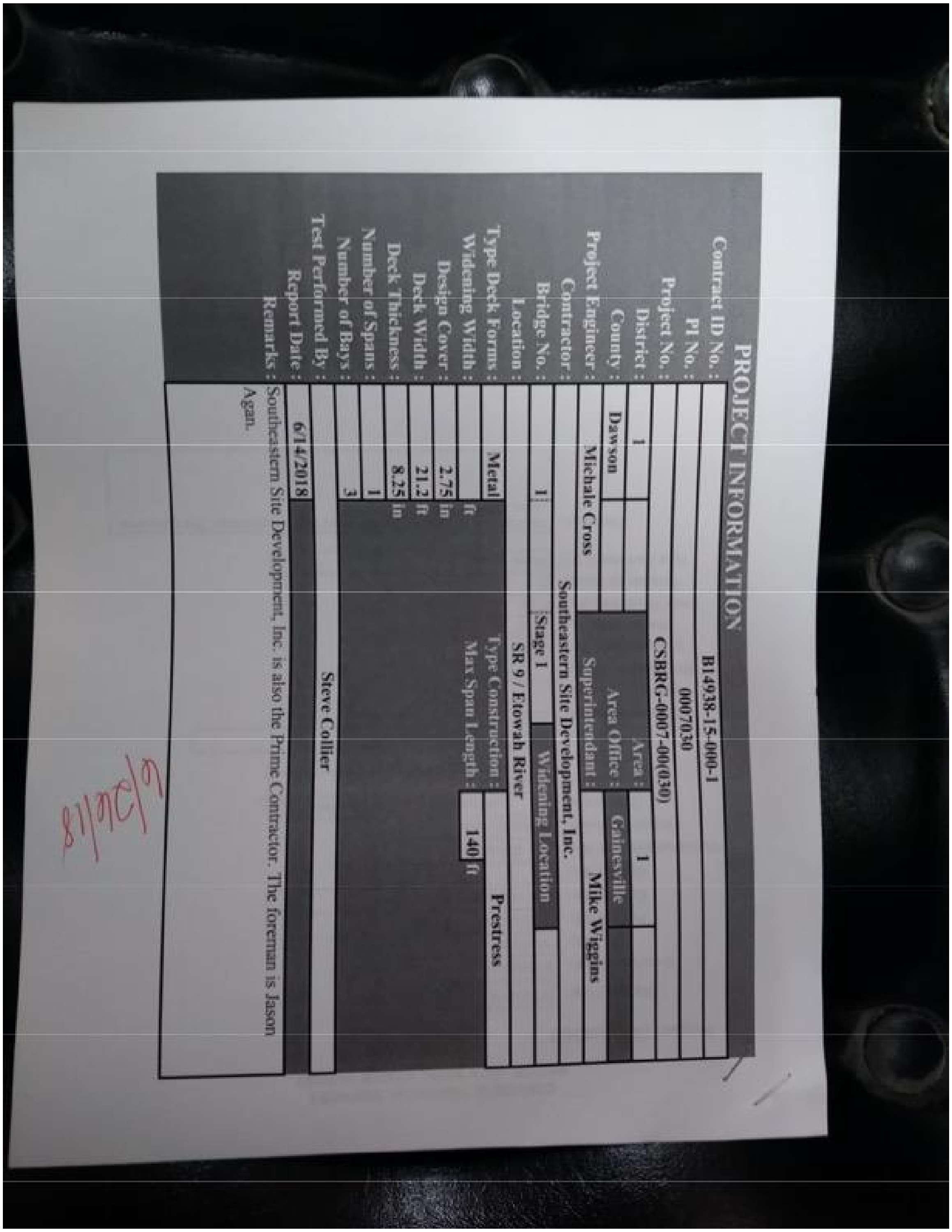
Attached are the distribution charts and/or drawings showing individual measurements and other data related to steel cover. All readings were obtained prior to surface corrections for smoothness, if required.

If you need additional information or if we may be of further assistance, please contact Gary Wood in the Concrete Branch at 404-608-4876.

MLF:GW:ars

Attachment

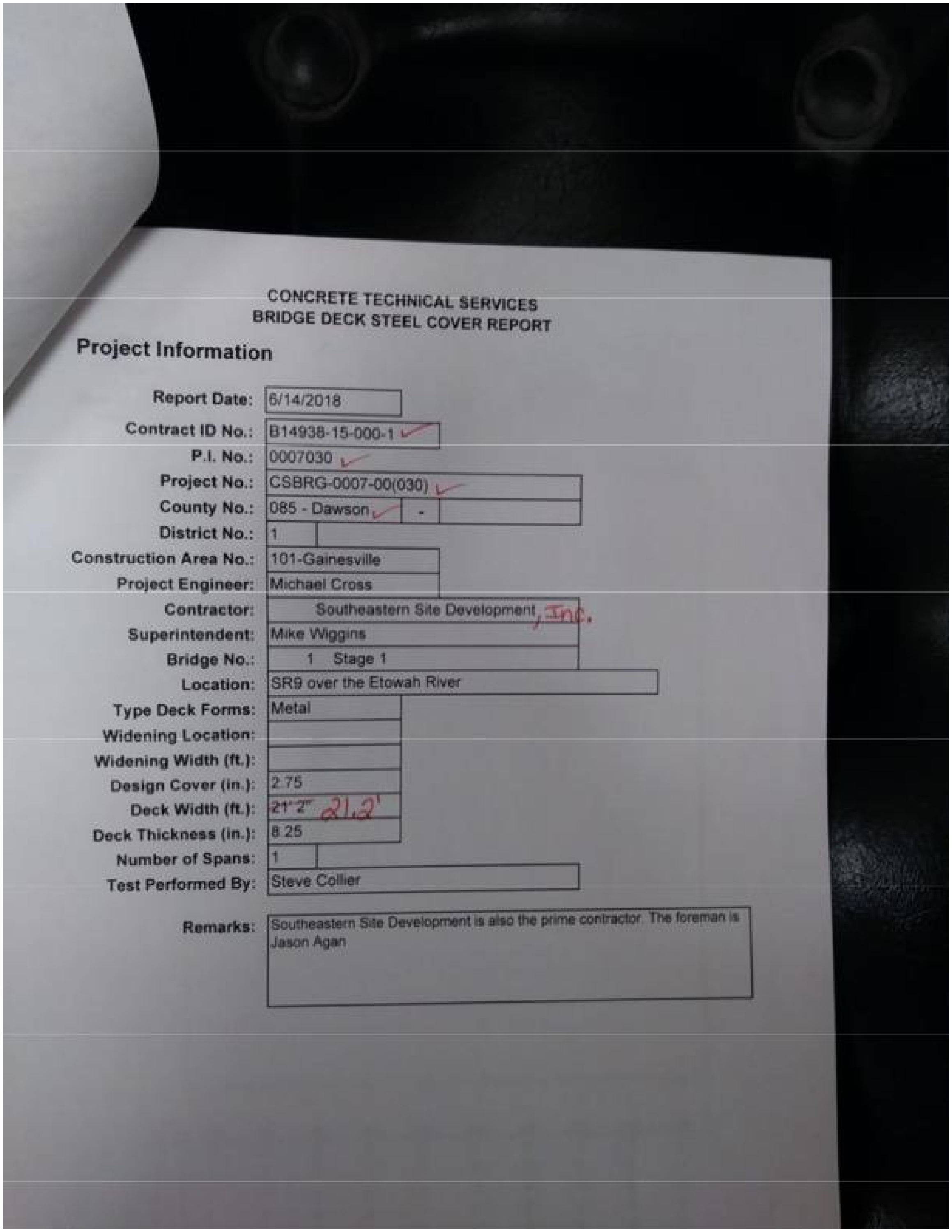
c: Marc Mastronardi, P.E., Director of Construction
John Hancock, P.E., State Construction Engineer
Andy Doyle, P.E., State Maintenance Engineer
Bill DuVall, P.E., State Bridge Engineer
Scott Frederick, Area Manager, Gainesville
Southeastern Site Development, Inc.



PROJECT INFORMATION

Contract ID No. :	B14938-15-000-1		
PI No. :	0007030		
Project No. :	CSBRG-0007-00(030)		
District :	1	Area :	1
County :	Dawson	Area Office :	Gainesville
Project Engineer :	Michale Cross	Superintendant :	Mike Wiggins
Contractor :	Southeastern Site Development, Inc.		
Bridge No. :	1	Stage I	Widening Location
Location :	SR 9 / Etowah River		
Type Deck Forms :	Metal	Type Construction :	Prestress
Widening Width :	ft	Max Span Length :	140 ft
Design Cover :	2.75 in		
Deck Width :	21.2 ft		
Deck Thickness :	8.25 in		
Number of Spans :	1		
Number of Bays :	3		
Test Performed By :	Steve Collier		
Report Date :	6/14/2018		
Remarks :	Southeastern Site Development, Inc. is also the Prime Contractor. The foreman is Jason Agan.		

6/26/18

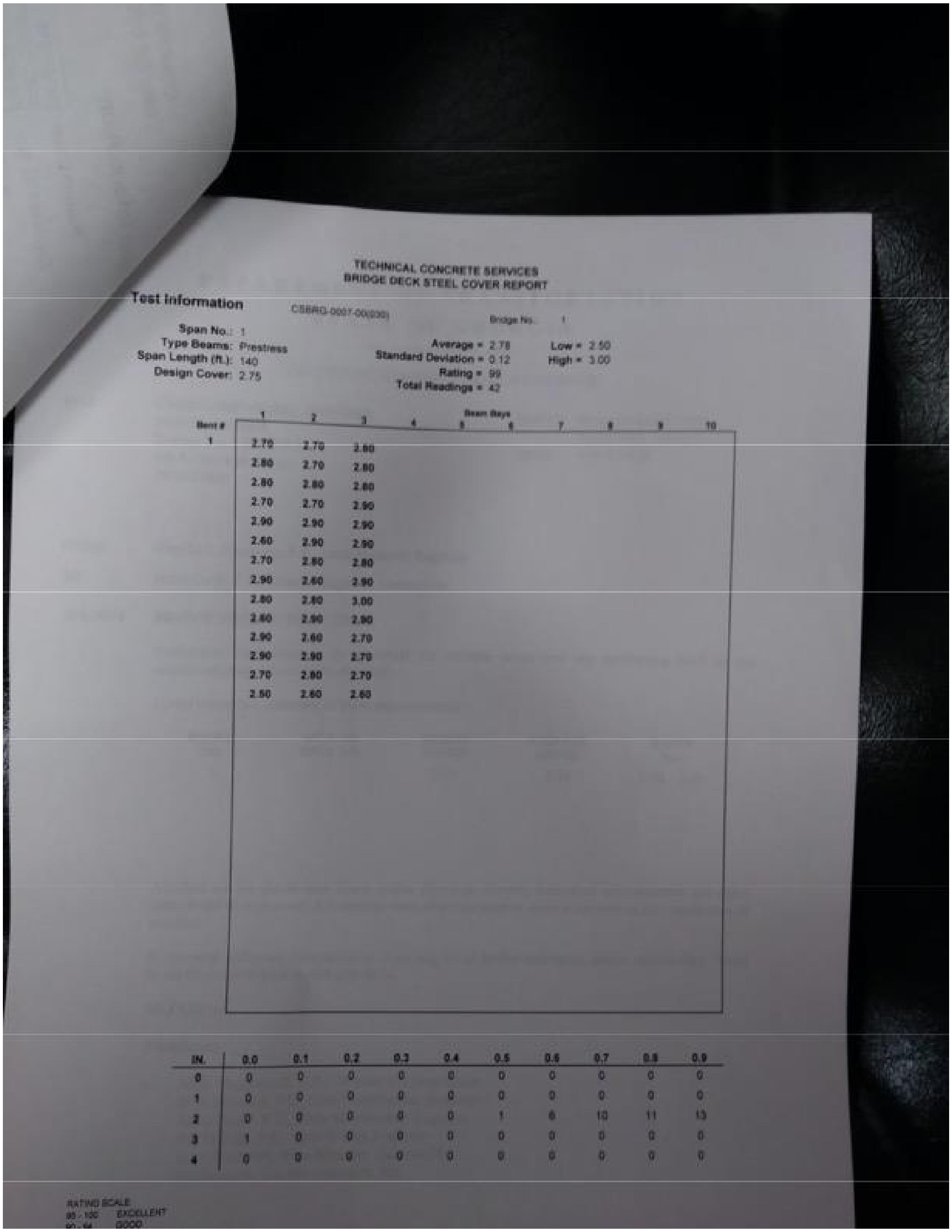


CONCRETE TECHNICAL SERVICES
BRIDGE DECK STEEL COVER REPORT

Project Information

Report Date:	6/14/2018
Contract ID No.:	B14938-15-000-1
P.I. No.:	0007030
Project No.:	CSBRG-0007-00(030)
County No.:	085 - Dawson --
District No.:	1
Construction Area No.:	101-Gainesville
Project Engineer:	Michael Cross
Contractor:	Southeastern Site Development, Inc.
Superintendent:	Mike Wiggins
Bridge No.:	1 Stage 1
Location:	SR9 over the Etowah River
Type Deck Forms:	Metal
Widening Location:	
Widening Width (ft.):	
Design Cover (in.):	2.75
Deck Width (ft.):	21' 2" 21.2'
Deck Thickness (in.):	8.25
Number of Spans:	1
Test Performed By:	Steve Collier

Remarks: Southeastern Site Development is also the prime contractor. The foreman is Jason Agan



TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information

CSBRG-0007-00(030)

Bridge No.: 1

Span No.: 1
Type Beams: Prestress
Span Length (ft.): 140
Design Cover: 2.75

Average = 2.78
Standard Deviation = 0.12
Rating = 99
Total Readings = 42

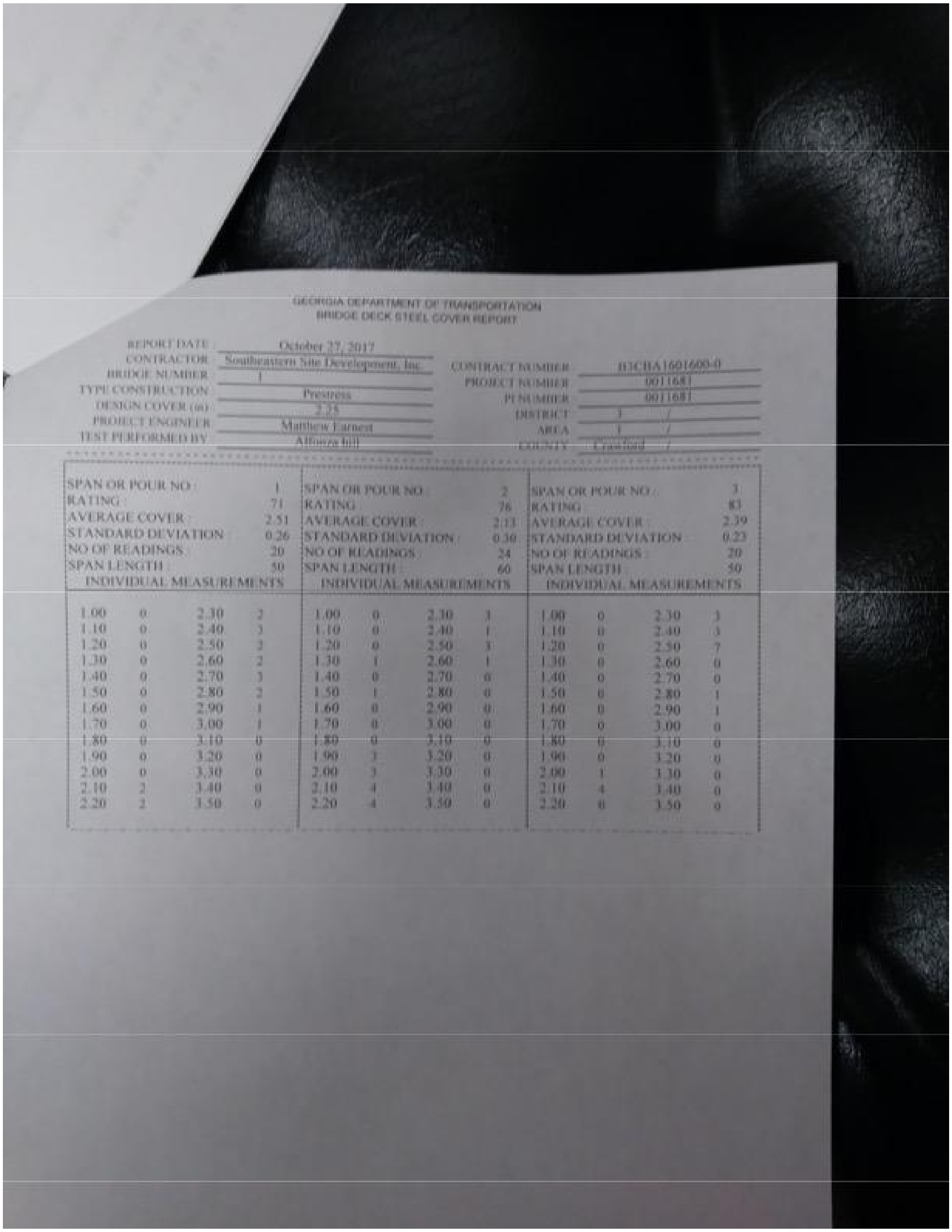
Low = 2.50
High = 3.00

Beam Bays

Bent #	1	2	3	4	5	6	7	8	9	10
1	2.70	2.70	2.80							
	2.80	2.70	2.80							
	2.80	2.80	2.80							
	2.70	2.70	2.90							
	2.90	2.90	2.90							
	2.60	2.90	2.90							
	2.70	2.80	2.80							
	2.90	2.60	2.90							
	2.80	2.80	3.00							
	2.60	2.90	2.90							
	2.90	2.60	2.70							
	2.90	2.90	2.70							
	2.70	2.80	2.70							
	2.50	2.60	2.60							

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	1	6	10	11	13
3	1	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
95 - 100 EXCELLENT
90 - 94 GOOD



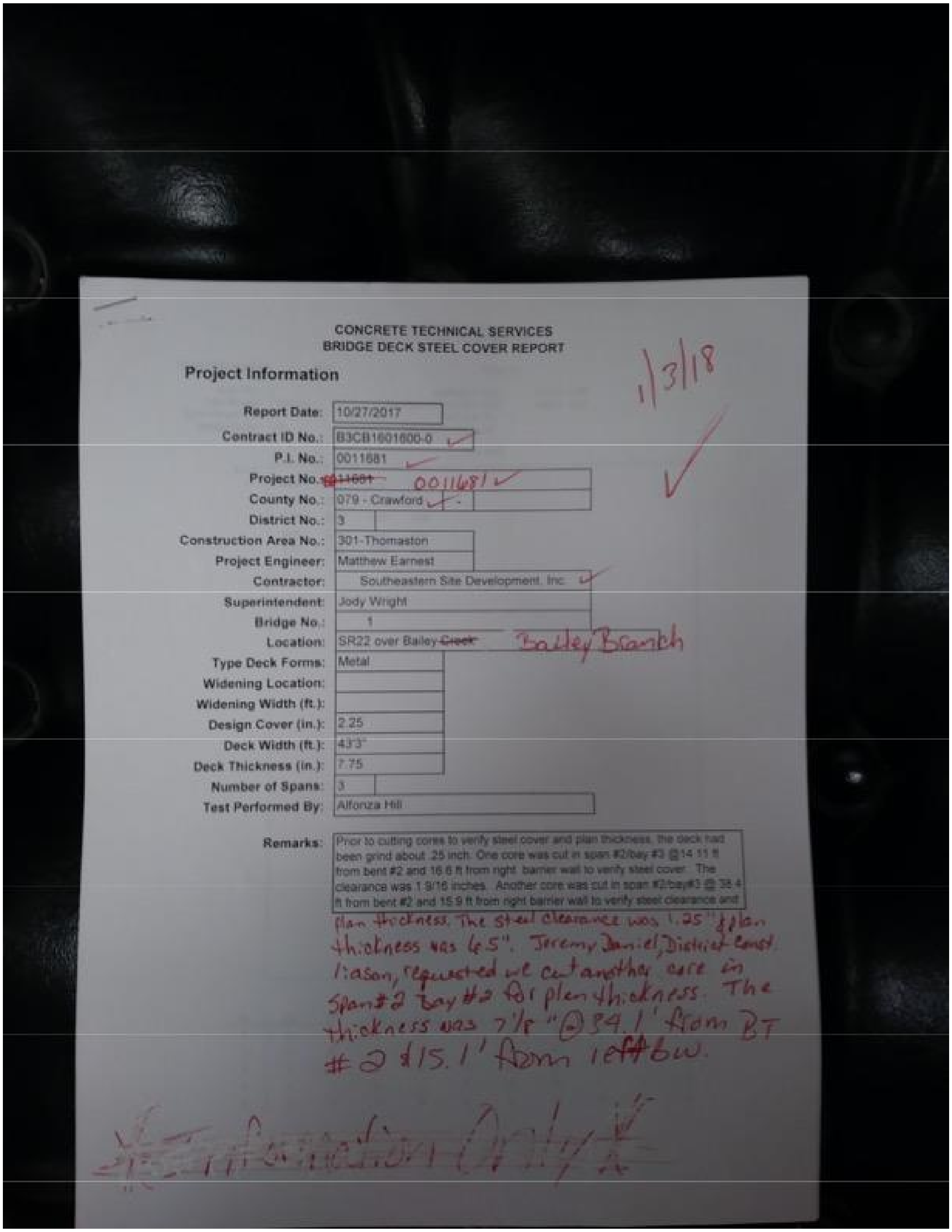
GEORGIA DEPARTMENT OF TRANSPORTATION
BRIDGE DECK STEEL COVER REPORT

REPORT DATE	October 27, 2017		
CONTRACTOR	Southeastern Site Development, Inc.	CONTRACT NUMBER	B3CBA1601600-0
BRIDGE NUMBER	1	PROJECT NUMBER	0011681
TYPE CONSTRUCTION	Prestress	PI NUMBER	0011681
DESIGN COVER (in)	2.25	DISTRICT	3 /
PROJECT ENGINEER	Matthew Earnest	AREA	1 /
TEST PERFORMED BY	Alfonza hill	COUNTY	Crawford /

SPAN OR POUR NO :	1	SPAN OR POUR NO :	2	SPAN OR POUR NO :	3
RATING :	71	RATING :	76	RATING :	83
AVERAGE COVER :	2.51	AVERAGE COVER :	2.13	AVERAGE COVER :	2.39
STANDARD DEVIATION :	0.26	STANDARD DEVIATION :	0.30	STANDARD DEVIATION :	0.23
NO OF READINGS :	20	NO OF READINGS :	24	NO OF READINGS :	20
SPAN LENGTH :	50	SPAN LENGTH :	60	SPAN LENGTH :	50

INDIVIDUAL MEASUREMENTS

1.00	0	2.30	2	1.00	0	2.30	3	1.00	0	2.30	3
1.10	0	2.40	3	1.10	0	2.40	1	1.10	0	2.40	3
1.20	0	2.50	3	1.20	0	2.50	3	1.20	0	2.50	7
1.30	0	2.60	2	1.30	1	2.60	1	1.30	0	2.60	0
1.40	0	2.70	3	1.40	0	2.70	0	1.40	0	2.70	0
1.50	0	2.80	2	1.50	1	2.80	0	1.50	0	2.80	1
1.60	0	2.90	1	1.60	0	2.90	0	1.60	0	2.90	1
1.70	0	3.00	1	1.70	0	3.00	0	1.70	0	3.00	0
1.80	0	3.10	0	1.80	0	3.10	0	1.80	0	3.10	0
1.90	0	3.20	0	1.90	3	3.20	0	1.90	0	3.20	0
2.00	0	3.30	0	2.00	3	3.30	0	2.00	1	3.30	0
2.10	2	3.40	0	2.10	4	3.40	0	2.10	4	3.40	0
2.20	2	3.50	0	2.20	4	3.50	0	2.20	0	3.50	0



CONCRETE TECHNICAL SERVICES
BRIDGE DECK STEEL COVER REPORT

1/3/18

Project Information

Report Date:	10/27/2017
Contract ID No.:	B3CB1601600-0
P.I. No.:	0011681
Project No.:	~~0011681~~ 0011681
County No.:	079 - Crawford
District No.:	3
Construction Area No.:	301-Thomaston
Project Engineer:	Matthew Earnest
Contractor:	Southeastern Site Development, Inc.
Superintendent:	Jody Wright
Bridge No.:	1
Location:	SR22 over Bailey ~~Creek~~ Bailey Branch
Type Deck Forms:	Metal
Widening Location:	
Widening Width (ft.):	
Design Cover (in.):	2.25
Deck Width (ft.):	43'3"
Deck Thickness (in.):	7.75
Number of Spans:	3
Test Performed By:	Alfonza Hill

Remarks: Prior to cutting cores to verify steel cover and plan thickness, the deck had been grind about .25 inch. One core was cut in span #2/bay #3 @ 14.11 ft from bent #2 and 16.6 ft from right barrier wall to verify steel cover. The clearance was 1 9/16 inches. Another core was cut in span #2/bay#3 @ 38.4 ft from bent #2 and 15.9 ft from right barrier wall to verify steel clearance and plan thickness. The steel clearance was 1.25" & plan thickness was 6.5". Jeremy Daniel, District Const. liason, requested we cut another core in Span#2 Bay #2 for plan thickness. The thickness was 7 1/8" @ 34.1' from BT #2 & 15.1' from left BW.

Information Only

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information 11681 Bridge No: 1

Span No.: 1
Type Beams: Prestress
Span Length (ft.): 50
Design Cover: 2.25

Average = 2.51
Standard Deviation = 0.27
Rating = 71
Total Readings = 20

Low = 2.10
High = 3.00

Beam Bays

Bent #				
1	2.30	2.50	2.10	2.60
	2.40	2.50	2.10	2.80
	2.20	2.40	2.20	2.80
	3.00	2.40	2.30	2.70
	2.90	2.60	2.70	2.70

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0
2	0	2	2	2	3	2	2	3	2	1
3	1	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
[illegible] EXCELLENT
[illegible] GOOD
[illegible] FAIR
[illegible] MARGINAL
[illegible] NEEDS IMPROVEMENT

TECHNICAL CONCRETE SERVICES
BRIDGE DECK STEEL COVER REPORT

Test Information 11681 Bridge No.: 1

Span No.: 2
Type Beams: Prestress
Span Length (ft.): 60
Design Cover: 2.25

Average = 2.13
Standard Deviation = 0.30
Rating = 76
Total Readings = 24

Low = 1.30
High = 2.60

Beam Bays

Bent #				
2	2.20	2.20	1.50	2.20
	2.30	2.10	1.30	2.30
	1.90	2.00	2.20	2.40
	2.10	2.10	1.90	2.50
	2.60	1.90	2.00	2.50
	2.50	2.00	2.10	2.30

IN.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	1	0	1	0	0	0	3
2	3	4	4	3	1	3	1	0	0	0
3	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0

RATING SCALE
85 - 100 EXCELLENT
GOOD

2. APPENDIX B: GDOT MIX DESIGN GUIDELINES

Reproduced from Section 500 of 2013 GDOT Standard Specifications Construction of Transportation Systems.

English								
Class of Concrete	(2) Coarse Aggregate Size No.	(1 & 6) Minimum Cement Factor lbs/yd^3	Max Water/ Cement ratio lbs/lb	(5) Slump acceptance Limits (in) Lower-Upper		(3 & 7) Entrained Air Acceptance Limits (%) Lower-Upper		Minimum Compressive Strength at 28 days (psi)
"AAA"	67,68	675	.440	2	4	2.5	6.0	5000
"AA1"	67,68	675	.440	2	4	2.5	6.0	4500
"AA"	56,57,67	635	.445	2	4	3.5	7.0	3500
"A"	56,57,67	611	.490	2	4	2.5 (3)	6.0	3000
"B"	56,57,67	470	.660	2	4	0.0	6.0	2200
"CS"	56,57,67 Graded Agg.*	280	1.400	-	3½	3.0	7.0	1000 (4)
metric								
Class of Concrete	(2) Coarse Aggregate Size No.	(1 & 6) Mini mum Cement Factor kg/m^3	Max Water/ Cement ratio kg/kg	(5) Slump acceptance Limits (mm) Lower - Upper		(3 & 7) Entrained Air Acceptance Limits (%) Lower-Upper		Minimum Compressive Strength at 28 days (MPa)
"AAA"	67,68	400	.440	50	100	2.5	6.0	35
"AA1"	67,68	400	.440	50	100	2.5	6.0	30
"AA"	56,57,67	375	.445	50	100	3.5	7.0	25
"A"	56,57,67	360	.490	50	100	2.5 (3)	6.0	20
"B"	56,57,67	280	.660	50	100	0.0	6.0	15
"CS"	56,57,67 Graded Agg.	165	1.400		90	3.0	7.0	7 (4)

Notes:

1. Portland cement may be partially replaced with fly ash as provided in Subsection 500.3.04.D.4 or with granulated iron blast furnace slag as provide for in Subsection 500.3.04.D.5.
2. Specific size of coarse aggregate may be specified.
3. Lower limit is waived when air entrained concrete is not required.
4. The mixture will be capable of demonstrating a laboratory compressive strength at 28 days of 1000 psi (7 MPa) + 0.18 R*. Compressive strength will be determined based upon result of six cylinders prepared and tested in accordance with AASHTO T 22 and T 126.

   * Where R = Difference between the largest observed value and the smallest observed value for all compressive strength specimens at 28 days for a given combination of materials and mix proportions prepared together.

5. Designed slump may be altered by the Office of Materials and Research when Type "F" water reducers are used.
6. Minimum cement factor shall be increased by 50 lbs/yd^3 (30 kg/m^3) when size No. 7 coarse aggregate is used.
7. When Class A is specified for bridge deck concrete, the entrained air acceptance limits shall be 3.5% to 7.0%.

Note: Office of Materials and Research is now Office of Materials and Testing (OMAT)

3. APPENDIX C: SELECT RAW DATA FROM DECOMMISIONED BRIDGES WITH DECK DEFECTS

Bridge #	Deck Area (ft^2)	Delamination/Spall/Patched Area (ft^2)			Abrasion/Wear (ft^2)		Cracking (RC and Other) (ft^2)		Efflorescence/Rust Staining (ft^2)		Exposed Rebar (ft^2)	
		State 2	State 3	State 4	State 2	State 3	State 2	State 3	State 2	State 3	State 2	State 3
1	9088						4460					
2	3445							10				
3	10098	265	1		4037		2108	2048				
4	2454					1312		486				
5	2412	1610	400		402							
6	3069				2913		135					21
7	108936		10		1400	2400	30	5				22
8	9116		11	20			1670	3150	298	3260		
9	8676		4392			2000		1500				400
10	3250				12		197	11				
11	8788				124		496	73	95			
12	9227						6714					
13	2056							20				
14	4590	61			1835		149		3			
15	3030							140				
16	8304		3231					3417				
17	10180						4000					
18	6919	11					43	45				
19	4554						55					
20	2208						20	546				
21	24000	110	62		21868		1600	360				
22	17629						70					
23	11366						30					
24	1446							800				
25	2430				650		150		75			
26	1666						1666					
27	11520						270					
28	5670				5250		420					
29	2800						2400					
30	806		6									
31	2015				1010		1005					
32	1612				808		804					
33	1209				606		603					
34	3390				1700		1690					
35	6600		40		70		4912		67			40
36	42452	2590			5246		33567		822			
37	21358	52			425		9295				26	
38	10935				240		561					
39	5090	10			450	1060		30				
40	12106						11100	645	361			
41	2400	28										
42	431					200		231				
43	11688						255					
44	15656							8000				
45	3831				1500		2100		231			
46	9576		1		3500	460	39					
47	3600					400	3200					
48	31220		168				28786					
49	31232	100	29				30313	740		12	30	8
50	2484						300					
51	1975						845	110				
52	2781	400	600				1000	781				
53	5221							78		26		
54	2814	36										
55	14475	15					2778					
56	2688				1200		1200					
57	3250				12		197	11				

RC and Other = In Reinforced Concrete and Other Materials

4. APPENDIX D: DRAFT STANDARD OPERATING PROCEDURE

Date

Georgia Department of Transportation
Office of Materials and Testing

Standard Operating Procedure (SOP)

Procedure for Calculating Pay Adjustments for Failing Concrete Cover Control of Reinforced Concrete Bridge Decks

I. General

It is the responsibility of the personnel from the Office of Materials and Testing (OMAT) to monitor the concrete cover control of all reinforced concrete bridges on Georgia Department of Transportation (GDOT) projects. If the concrete cover control is inadequate and cannot be remediated, the project engineer is empowered to reject the work as outlined in Section 105.12 of the Standard Specifications. The purpose of this SOP is to provide a means of calculating pay reductions for failing concrete cover control of reinforce concrete bridge decks that in the project engineer's judgement do not warrant outright rejection of the work.

To inform the project engineer as to the cover control of the reinforced concrete bridge deck, a series of no fewer than 20 cover measurements will be taken per span. The method of sampling may either be a systematic sampling on a grid with approximately 10 ft separations, or any manner of random sampling. The method of executing the cover measurements is set forth in British Standard 1881-204, and is to be observed. The results from these measurements will be used to calculate the pay reduction factor for each span as outlined in the method below.

A. Method of Calculating Pay Reduction For Failing Cover Control

The pay reduction will be determined by the specified pay factor equation below.

$$PF=55+0.5(PWL) \qquad \text{Eq.1}$$

In the equation, PF is the pay factor for the span, and PWL is the estimated percent of the true cover distribution that is within the limits in Table 1, based on the specified design cover.

Table 1. Control Limits for Common Design Covers

Design Cover (in)	Lower Cover Limit, LCL (in)	Upper Cover Limit, UCL (in)
2	1.75	2.25
2.25	2.00	2.50
2.5	2.25	2.75
2.75	2.5	3.00

To calculate the PWL, the following procedure is followed. First, the sample mean and standard deviation of the 20 randomly sampled cover measurements is calculated. If the sample mean is outside the limits as provided in Table 1, the work should be rejected and remediated. Provided that the sample mean is within the limits from Table 1, the upper and lower Q indices are to be calculated according to these equations:

$$Q_U = \frac{UCL - Sample\ mean}{Standard\ Deviation} \qquad \text{Eq.2}$$

$$Q_L = \frac{Sample\ mean - LCL}{Standard\ Deviation} \qquad \text{Eq.3}$$

Where the UCL and LCL are taken from Table 1. Next, the PWL must be determined for both the upper and lower Q indices. To determine the PWL, the value of the Q index is matched to Table 2 below (for n=20), noting that if the Q value falls between table values, to round up to the next PWL.

Table 2. PWL Reference

PWL	Q	PWL	Q	PWL	Q	PWL	Q	PWL	Q
100	3.20	89	1.22	78	0.78	67	0.45	56	0.15
99	2.18	88	1.17	77	0.75	66	0.42	55	0.13
98	1.96	87	1.12	76	0.71	65	0.39	54	0.10
97	1.81	86	1.08	75	0.68	64	0.36	53	0.08
96	1.70	85	1.04	74	0.65	63	0.34	52	0.05
95	1.61	84	1.00	73	0.62	62	0.31	51	0.03
94	1.52	83	0.96	72	0.59	61	0.28	50	0.00
93	1.45	82	0.92	71	0.56	60	0.26		
92	1.39	81	0.88	70	0.53	59	0.23		
91	1.33	80	0.85	69	0.50	58	0.20		
90	1.27	79	0.81	68	0.47	57	0.18		

Next, the PWL for the upper and lower bound are combined into a single PWL according to the equation below:

$$PWL = (PWL\ upper + PWL\ lower) - 100 \qquad \text{Eq.4}$$

Next, the PF is calculated according to Eq. 1, using the PWL from Eq. 4. Finally, the value of the pay factor will be used to calculate the payment for the span according to the equation below:

Adjusted Payment=(PF/100)*(Original Payment) Eq.5

B. Example Pay Reduction Calculation

Suppose a bridge contains only one span, with a design cover of 2.25", and an original payment of $100,000. In accordance with the method in section A, 20 cover measurements are randomly taken which are found to be as follows:

Table 3. Results of the Randomly Selected Cover Points

Location	Cover Value (in)	Location	Cover Value (in)
1	2.10	11	2.00
2	2.30	12	2.20
3	2.20	13	2.30
4	1.90	14	2.50
5	1.80	15	2.90
6	2.10	16	2.10
7	2.50	17	2.60
8	2.60	18	2.30
9	2.00	19	2.20
10	2.30	20	2.30

From Table 3, the lower cover limit is 2" and the upper cover limit is 2.5" (inclusive). The sample mean and standard deviation are found to be 2.26" and 0.258", respectively. Using Eqs. 2 and 3, the Q indices are found to be 0.93 and 1.01 for the upper and lower bounds respectively. Using Table 2 and selecting the next largest PWL when the Q index falls between table values, the PWLs are found to be 83 and 85. The combined PWL is then found to be 68 (83+85-100). Using Eq.1, the pay factor is then found to be 89%. Therefore, the adjusted payment is thus found to be (89/100)*$100,000, which equates to $89,000. For this example which only consists of one span, the adjusted payment is $89,000 for the bridge deck.

Suppose that instead of a standard deviation of 0.258", the standard deviation was calculated as 0.129" instead, with the same sample mean of 2.26". In that case, the Q indices are 1.86 and 2.02 for the upper and lower indices respectively. Using Table 2, those Q indices would yield PWLs of 98 and 99. The combined PWL would then be calculated as 97. Using Eq.1, the pay factor is then found to be 103.5%. Therefore, the adjusted payment is thus found to be (103.5/100)*$100,000, which equates to $103,500. In this case, since the mean was almost exactly the design value, and the standard deviation was reduced by half, the bridge contractor received a reward for the additional performance expected out of this bridge span.

Note: Pay may be capped at original contract price without bonus, even if the pay factor is greater than 100%, due to limited project budget.

II. Report

The Office of Materials and Testing will provide a letter of recommendation to the District Engineer to include a pay factor reduction or a waiver for all failing cover control projects. The Director of Construction, State Construction Engineer, Area Engineer, and OMAT's Material Audits Unit will be copied on all letters of recommendation.

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State Materials Engineer

____

Director of Construction

ACKNOWLEDGEMENTS

The following individuals at GDOT provided many valuable suggestions throughout this study: Mr. Jeremiah Daniel, State Bridge Construction Liaison; Mr. Bob O'Daniels, Bridge Asset Manager; Mr. Jason Waters, Concrete Branch Supervisor; Mr. Peter Wu, Bureau Chief; and Ms. Supriya Kamatkar, Assistant Office Head, Office of Performance-based Management and Research.

The opinions and conclusions expressed herein are those of the authors and do not represent the opinions, conclusions, policies, standards, or specifications of GDOT or of other cooperating organizations.

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The authors express their profound gratitude to all of these individuals for their assistance and support during the completion of this research project.

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