Quantifying Bridge Deterioration Catalysts Through Environment Classifications

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APPROVAL SHEET

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ABSTRACT

Many bridges in the Commonwealth of Virginia are rapidly approaching the end of their designed service life and proper management of these structures has become a priority for the Virginia Department of Transportation. Historical bridge inspection records and GIS are used to identify environments associated with increased bridge deterioration. Markov chains and Weibull distributions models are used to study the relative impact an environment has on bridge element deterioration.

In this investigation, eleven environment classifications relating to a bridges district ownership, traffic levels, bridge geometry, and weather exposure are identified and the resulting deterioration effects are quantified in terms of modification factors compatible with AASHTOWare Bridge Management. Implementation of modification factors was found to provide higher accuracy deterioration models than the models currently used by the Department of Transportation.

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INTRODUCTION

Strategic management of rapidly aging infrastructure has become a critical task for agencies across the United States. The Virginia Department of Transportation 2019 budget allocates roughly $2.2 billion, or 40.7% of its annual budget towards road maintenance and operations (VDOT, 2018). Of these funds, $200 million is typically directed towards bridge maintenance and an additional $200 million of the Departments budget is put towards bridge construction. Currently, the average age of bridges in Virginia is 48 years, yet many of these bridges were designed for a service life of only 50 years (Bales, Chitrapu, & Flint, 2018). With more than 21,000 bridges in its inventory (VDOT, 2019) proper allocation of funds towards maintenance and repair is crucial to maintain a safe and operational transportation network.

This aging and subsequent deterioration has not gone unnoticed by the American people. The American Society of Civil Engineers (ASCE) publishes an Infrastructure Report Card every four years that assess the condition of key American infrastructure components ranging from solid waste to aviation. One of the categories studied in this well publicized report are bridges. The 2015 Infrastructure Report Card gave Virginia a "C" rating. The report states that, "Available funds are often used to address immediate repair or replacement needs, leaving few remaining funds for preventative maintenance" (ASCE Government Relations, 2015). As such, VDOT and many other DOT's are beginning to taking active measures to identify ways to more effectively allocate their resources.

The Virginia Department of Transportation has been working to solve this problem by closely monitoring and studying the deterioration of its bridges, the foundation of a bridge management system. By studying trends in bridge deterioration the Department is able to extend the expected lifespan of existing bridges and maintain a user friendly network of roadways throughout the state.

History of Bridge Management Systems

The 1950s and 1960s saw a major increase in bridge construction across the United States. During this period of economic boom, the construction industry and society as a whole were enjoying the new higher standard of living and freedom of mobility that bridges and transportations systems offered. This mentality quickly came to halt in 1967 when the 39-year-old Silver Bridge in West Virginia collapsed under the weight of rush-hour traffic and made national headlines. Investigations revealed that the collapse may have been avoided if formal inspection and maintenance policies had existed so that aging infrastructure could be managed systematically. With the support of the public, the National Bridge Inspection Standards were implemented in 1971 (Ryan, Mann, Zachary, & Ott, 2012).

Over the following decades, much advancement was made to standardize inspection practices and to better maintain bridge inventories. In 1991, the Federal Highway Administration (FHWA) helped develop a comprehensive bridge management system called Pontis. Pontis provided agencies with many of the tools needed to catalog and study bridge inspection reports. Amongst other things, Pontis allowed agencies to track the health of individual bridges over its entire lifespan and take proactive steps to repair bridges and save tax-payer money (Ryan, Mann, Zachary, & Ott, 2012). After several software updates and redesigns, Pontis was renamed as AASHTOWare Bridge Management (BrM) in 2015 and is utilized by over 40 state Departments of Transportation (AASHTO, 2013a). Bridge management systems rely heavily on predictive deterioration models in order to provide useful insight on the health of bridges in the future. AASHTOWare BrM utilizes a hybrid deterioration model consisting of both a Markovian and a Weibull model to predict future condition states (AASHTO, 2019). When calibrating the BrM software, the user has the option to assign model parameters to individual bridge elements to reflect the behavior of their own bridge inventory. Because BrM is used to determine where and when to spend funding, properly calibrating models to reflect local bridge conditions and features can save Virginia significant amounts of money (VDOT, 2016).

To populate the BrM database two important documents are frequently used - the Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges (FHWA, 1995) and the Specification for the National Bridge Inventory (NBI) Bridge Elements (FHWA, 2014). The FHWA Coding Guide is used to document basic information such as a bridges identification number, location and year built as well as design based information such as number of lanes, design load, and road width. The NBI is used to characterize the condition state or “health” of individual bridge components during each inspection event. The NBI has created a uniform numbering system to reference individual bridge components, or elements. Table A1 in Appendix A provides the element numbers for many commonly found bridge elements. During an inspection, each bridge elements’ observed condition state is described terms of four predefined condition states. Condition State 1 implies good health, Condition State 2 – fair health, Condition State 3 – poor health, and Condition State 4 – severe health. The quantity of an element that is in each condition state is reported during each inspection (AASHTO, 2013b) and used to define an overall condition state of the bridge.

Concrete Deterioration

Concrete is one of the most practical structural materials used in modern construction. Its low material cost and considerable compressive strength makes it an ideal candidate for many applications in bridge construction. While sometimes used for cast-in-place girders, concrete can almost always be seen as the primary building material for bridge abutments, columns, pier caps, and decks. The structural members of bridges are constantly being exposed to extreme loading from heavy traffic which can lead to cracking of the concrete. With time, capillary action will draw water to the reinforcing steel within the concrete elements. The introduction of moisture and oxygen to the reinforcing steel inevitably leads to corrosion. The steel corrosion causes two primary issues. First, the loss of reinforcing steel can reduce the strength of the element potentially to an unsafe condition. Secondly, the volume of rust that is created is around six time larger than the volume of steel that was affected (NACE International, 2012). The increase in volume is restricted by the surrounding uncracked concrete resulting in stress concentration that eventually give way to additional cracking or spalling of the concrete.

Concrete bridge elements are exposed to drastic temperature changes on an annual and daily basis as the sun heats bridge elements throughout the day and then temperatures fall at night, with additional thermal variations across the seasons. As concrete goes through heating and cooling cycles it will expand and contract creating a cycle of increased stress that can adversely affect the health and internal structure of a concrete element. When a nondurable aggregate is used the bond created by the cement can break down and result in cracking (Lwin, 2012). Also with cooler weather comes snow and ice. To maintain a safe driving surface, harsh deicing salts are regularly applied to bridge decks. The introduction of these salts may further increase the deterioration of several concrete elements.

Steel Deterioration

In lieu of a vehicle strike or an instantaneous loading event, a primary means of steel degradation is through corrosion. Steel bridges come into contact with water frequently via rainfall but are also subjected to splashing when roadways pass beneath the structure. The continuous repeated splashing and ponding that occurs through deck penetration creates a significantly harsh environment that increases corrosion rates (Kayser & Nowak, 1989). A high presence of de-icing salts can further increase these rates to those similar to marine environments (Ghodoosipoor, 2013). Nationwide, it is estimated that the direct costs of corrosion on highway bridges is $8.3 billion annually (Koch, Brongers, Thompson, Virmani, & Payer, 2002).

Non-commercial pedestrian vehicles are relatively light compared to commercial semi-trucks and it has been studied that heavy truck traffic is the cause for nearly all fatigue damage of steel bridge girders (Moses, Schilling, & Raju, 1987). Over time the repeated cyclical loading and unloading of steel girders will result in the development of microscopic cracks near the surface of the girder (Kim & Laird, 1978). These microscopic cracks can lead to sudden and unexpected failure if left untreated.

PURPOSE AND SCOPE

Motivation

Managing a detailed inventory of a state’s bridges and pavement systems is essential for the forecasting of immediate and future construction projects. With billions of dollars and public opinion at stake, agencies need to support their decision-making process with empirical evidence. The Moving Ahead for Progress in the 21st Century Act (MAP-21) provides a major avenue for the funding of surface transportation programs, but requires states to create a formal Transportation Asset Management Plan (TAMP). Each TAMP is required to provide a comprehensive list of assets, a description of each assets current condition, asset management objectives and measures, identification of performance gaps, analysis of lifecycle cost and risk management, financial plans, and investment strategies. The overall goal of a TAMP is to create a formal plan for asset management to meet the required levels of service and performance targets in the most cost-effective way (Transport Scotland, 2007). Bridge management software, such as AASHTOWare BrM, help congregate historical asset data to make the development of TAMP’s possible. By studying the available data, general trends can be discovered and shortcomings can be improved upon.

The practice of collecting and studying bridge inspection records for the purpose of asset management is still in its infancy. Many inspection and record keeping techniques are rapidly evolving to keep up with industry requirements. These changes however often result in compatibility issues when using older coarse scale data with modern fine-scale modeling techniques. Several agencies across the country have been slow to adopt in-depth bridge inspection programs and can lack the volume of data needed to optimize an asset management plan.

Purpose of deterioration modeling

A better insight into the deterioration rates of a state's bridges allows for changes to be made at both a broad and narrow scale. On a broad scale, agency-wide decision making strategies and design preferences can be updated if it is found that particular materials, construction assemblies, or operational environments tend to deteriorate faster than others. Updating design standards based on these types of findings could reduce maintenance costs in the long run. On a narrow scale, individual bridges can be compared to determine where funds are needed the most. The ability to forecast the future health of individual bridges in the inventory is a major key for preventative maintenance and determining planned strategies for cost-effective treatments compatible with the state's TAMP. A key parameter in this forecasting approach is understanding the rate of change of deterioration and performance for an owner's inventory.

Global and Local Environment Factors

To increase the accuracy of BrM deterioration models in Virginia, numerous studies have been completed to determine transition rates that most accurately reflect the behavior of Virginia's bridges subjected to a particular environment. It is common practice to establish deterioration parameters for subsets of bridge elements. For instance, a large database consisting of historical deterioration rates of bridge decks could be subdivided by bridge deck type, i.e. cast in place concrete, timber decks, etc. Similar subcategories could be developed for other bridge elements such as girders, bearings, abutments, etc.

In addition to material based classifications, it is reasonable to assume that a large catalog of bridges can be subdivided into smaller groupings where deterioration parameters would exhibit less variance based on other common variables. When investigating a similar topic for bridges in the state of Nevada, it was found that bridges exposed to more frequent freeze-thaw cycles and those exposed to heavier volumes of deicing salts deteriorated at a higher rate than those excluded from those group (Sanders & Zhang, 1994) (Mauch & Madanat, 2001) .

In general, different environmental conditions can be described as either a global or local environment. Global environmental factors are those that occur at a regional level or simply could be grouped together on a map by encircling a continuous area based on a measurable characteristic. Examples of global environmental factors include, but are not limited to, DOT jurisdictions, temperature exposure, proximity to the coast, or average daily traffic. Local environmental factors are described as factors relating to a bridges construction method or relative geometry. The presence of local environment factors will often need to be verified during bridge inspections. The presence or absence of joints or the subjectivity to being splashed by vehicular traffic below the structure are both examples of local environment factors.

Effective Modification Factors

AASHTOWare BrM allows for the modification of deterioration rates based on the presence of environmental factors (AASHTO, 2014) through the use of an *adjustment factor* as follows:

$f = f^{E} * f^{F} * f^{M}_{combined}$ where, [Eqn. 1], (AASHTO,2014 Eqn. 13)

f is the adjustment factor
f^{E} is the environment factor
f^{F} is a formula factor estimated from a user-customized formula
$f^{M}_{combined}$ is the combined modification factor for all protective systems

$M_i' = f * M_i, \forall i$ where, [Eqn. 2], (AASHTO,2014 Eqn. 14)

M_i' is the adjusted median years to transition for state i
f is the adjustment factor
M_i is the typical median years to transition for state i

While the formula is designed to utilize a single environmental factor, the formula factor, f^{F}, is permitted to take into account multiple parameters. This freedom to incorporate multiple factors has the potential to increase the predictive capability of BrM deterioration models. While the software provides the framework for the models to take place, BrM leaves it to the agency to determine the modification factors to be used. To determine realistic values in-depth studies of various local and global environments must be studied and compared.

Markov and Weibull Deterioration Models

Markov models were developed by Russian mathematician Andrei Markov and have since become a mainstay in a variety of fields such as meteorology, biology, and chemistry (Gagniuc, 2017). A properly calibrated Markov model will statistically predict future state of a stochastic process by observing only the current state. This principle that a future state relies solely on the current state, and not any previous state is known as the Markov property and often is referred to as being “memoryless” (Gagniuc, 2017). A Markov model is defined by a transition matrix which describes the probability that an observed element will either stay in its current state or transition into a separate state over a set time period.

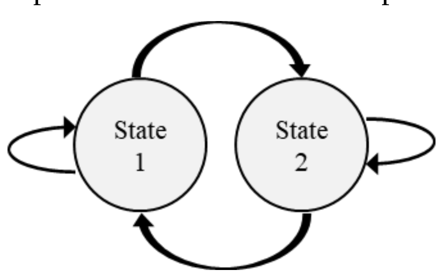

Figure 1. Markov Transitions

An alternative to Markov models is the Weibull distribution model. Weibull distributions are frequently used in reliability studies to evaluate the lifespan of a product or to determine the probability that a product will fail at any given point in time (Riveros & Arredondo, 2010). Weibull models are defined with “alpha” and “beta” parameters which control the shape of the deterioration curve (Riveros & Arredondo, 2010) as seen in Figure 2.

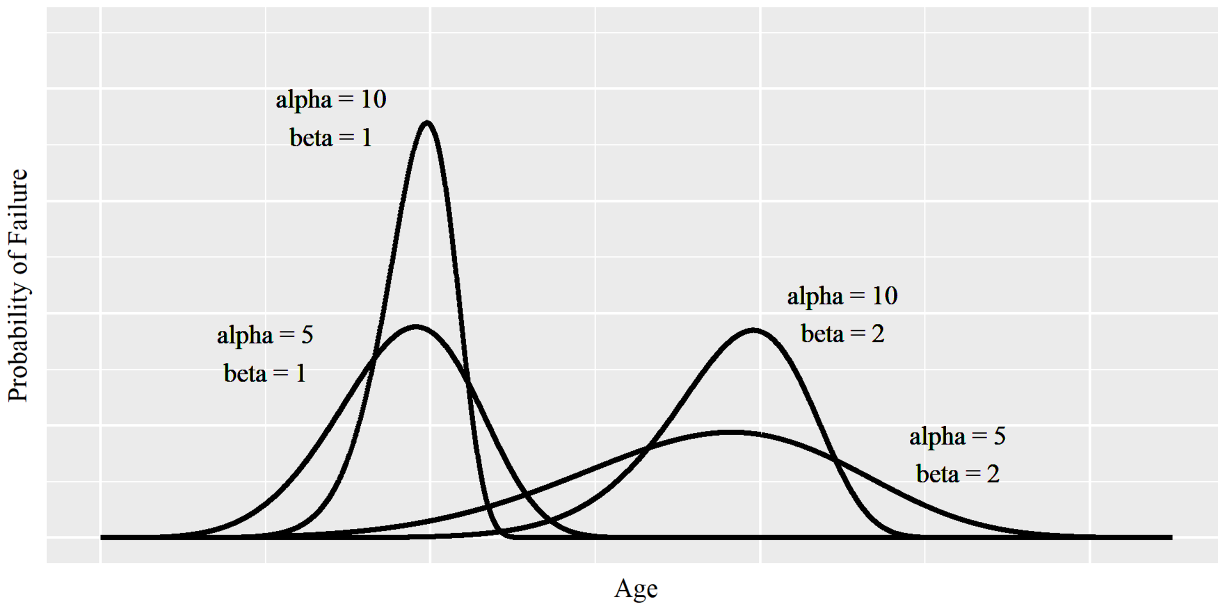

Figure 2. Weibull Distribution

The selection of an appropriate model whether Markov, Weibull, or any other, is largely dependent on the behavior exhibited by the data itself. This study uses a Weibull distribution to model the length of time required for bridge deterioration to begin and Markov models to predict the rate of continued deterioration. In any case, forecasting of future condition states is reliant on using the most current data available and model accuracy can be greatly improved by the addition of new data points. As such, it is important for models to be reevaluated frequently as new inspection records are acquired (Kleywegt & Sinha, 1994) (Bu, 2013).

Element Level Deterioration Rates

Modeling of structural elements has historically been done at either a “general” or “specific” level scale (Sanders & Zhang, 1994). A general model predicts the deterioration of either the substructure or superstructure as a whole. Whereas a specific or element level study would involve the individual bridge element. For example, an element level study might look at the deterioration of prestressed concrete piles whereas a general level study would analyze piles, abutments, and girders of all materials at once. As bridge management practices have evolved and become more sophisticated the ability to model individual bridge elements has become feasible allowing comprehensive element level models to be implemented on a broad scale. However, while the capability exists compatibility of useable data is not always available.

METHODS

Data Collection

In compliance with FHWA bridge inspection requirements, bridges are to be inspected at a minimum every 48 months (Code of Federal Regulations, 2011). While conducting bridge inspections, the FHWA Coding Guide is used to determine the quantity of each bridge element that is in Condition State 1, 2, 3, and 4. Bridge inspection reports are then uploaded to Virginia's BrM software and analyzed to determine the best use of funding and other limited resources. The data used for this study comes from a data extraction of BrM that provides element level inspection data collected up to February 10, 2016.

During each of the studies documented in the following sections, element condition state records are reported from the database using Microsoft Access queries. Included in each query is the bridge's age at the time of inspection and details about each bridge's local and global environment. Following a preliminary data filtering process, bridge records are grouped together based on age and used to model and compare deterioration rates based the bridges environment.

Data Filtering

Virginia Department of Transportation is not responsible for all bridges located within the state. Many small bridges are either privately owned or owned and maintained by an independent jurisdiction. VDOT has developed a Virginia Responsible Structures list used to differentiate which bridges are to be reported to the FHWA for funding and documentation purposes. NBI bridges are coded as 1 and non-NBI bridges are coded as 2. For the purpose of this study bridges coded as either 1 or 2 will be analyzed.

As the purpose of this study was largely to determine the unimpeded deterioration rates of bridge elements, bridges that were known to have undergone major reconstruction were also eliminated from the study. This was done by using Federal Item 106 which indicates the year in which a bridge was considered "reconstructed". When a year was listed in this field the bridge was consequently removed from the dataset. Some inspection records were entered incorrectly into the database resulting in bridge aged less than zero or ages significantly older than they actually are. To account for this only bridges whose age is between 0 and 100 years old were retained for the studies. Figure 3 details the current age of bridges in the database. The vertical line at 47.5 years denotes the average bridge age as of 2016.

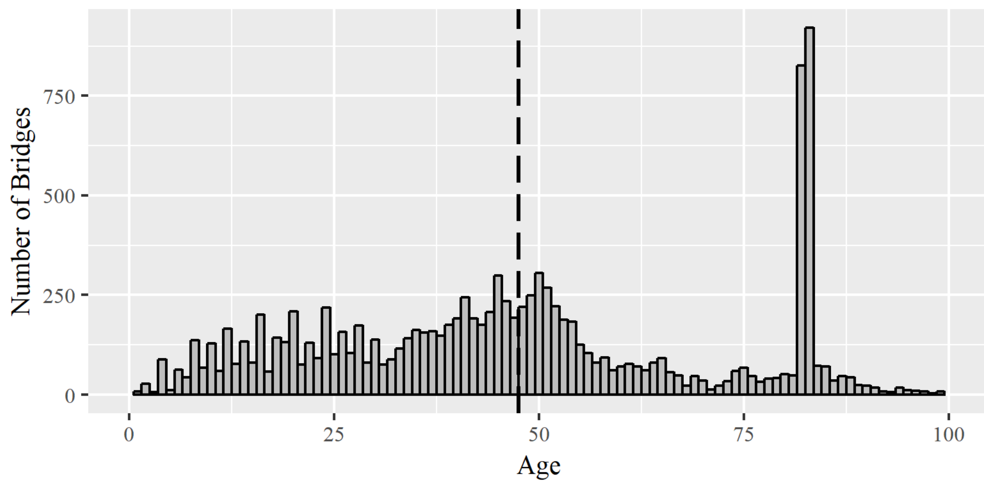

Figure 3. Current Age of VDOT Bridges

The histogram reveals a large number of bridges aged 82 and 83 years, this is likely an artifact of inaccurate reporting of a bridge's original year of construction. Because this study focuses on bridges below the age of 50, this error will not cause an impact. Figure 4 shows the quantity of historical inspection records available for bridges at the time of each inspection.

Figure 4. Age Distribution of Bridge Inspection

Figure 4 shows a distinct bimodal distribution of available bridge records. A portion of the second mode is attributed to the error in bridge construction year seen in Figure 3. 68% of the available inspection records are of bridges aged 0 to 50.

Low Level Maintenance and Inspector Variability

Under normal operating conditions (traffic, temperature changes, inclement weather, etc.), it would be expected that a bridge will continuously deteriorate from Condition State 1 to Condition State 4 sequentially and in the absence of any maintenance would never increase in health. However, routine low level maintenance may superficially make an element appear to be in slightly better health than what was reported during a previous inspection. Additionally, even with guidelines set by VDOT and the FHWA the bridge inspection process is still largely subjective and inspectors may have resistance to documenting a condition state worse than a previous inspection. Because there are many VDOT inspectors and outside consultants brought in to inspect bridges there is likely to be some level of variation in element level condition ratings depending on the abilities of the bridge inspector. Previous studies have identified the visual acuity, color vision, workloads, fear of traffic, and other variables all lead to significant variation in bridge condition state assessments (FHWA, 2001).

To model the unimpeded deterioration, significant increases in bridge health should be removed from the study. A naive approach to cleaning the data is to exclude any such records where the element quantity in Condition State 1 (CS1) increases between inspection years. However this method can remove excessive data points due to the noise created by low level maintenance and inspector subjectivity. This method also does not adequately handle changes in the proportion of an element that is in CS2, CS3, or CS4.

Three improvement filters were created in order to study the impact of removing data subjected to different allowances of low level maintenance. Each of the filters were run on an identical dataset of reinforced concrete bridge abutments (element 215) of all bridges state-wide. Table 1 details the methods used in each filter.

Table 1. Improvement Filters

Method	Rule	Remove record if:
1	1	No improvement filtering applied except for exclusions detailed in the previous section
2	1	Percentage of data in Condition State CS 1 increases
	2	Percentage of data in CS 4 decreases
3	1	Percentage of data in CS 1 increases by more than X%*
	2	Percentage of data in CS 4 decreases by more than X%*
4	1	Percentage of data in CS 1 increases by more than X%* between inspections
	2	Cumulative percentage of data in CS1 and CS2 increases by more than 2(X)*% between inspections
	3	Percentage of data in CS4 decreases by more than X%* between inspections
*Note: Where X% is used, models were run for X = 1, 3, 5		

The fictional data presented in Table 2 details the application of method 4 using X = 3%.

Table 2. Improvement Filter Example

Bridge I.D.	Inspection	CS1	CS2	CS3	CS4	Note
000000000000001	**i**	90%	8%	2%	0%	-
000000000000001	**i+1**	88%	8%	3%	1%	-
000000000000001	**i+2**	92%	5%	3%	0%	Rule 1
000000000000001	**i+3**	88%	7%	3%	2%	-
000000000000001	**i+4**	85%	13%	1%	1%	Rule 2
000000000000001	**i+5**	85%	7%	4%	4%	-
000000000000001	**i+6**	85%	7%	7%	1%	Rule 3
000000000000001	**i+7**	85%	7%	5%	3%	-

By removing inspection records that violated the rules, the data set for this particular bridge is reduced and shown in Table 3.

Table 3. Improvement Filter Example

Bridge I.D.	Inspection	CS1	CS2	CS3	CS4
000000000000001	**i**	90%	8%	2%	0%
000000000000001	**i+1**	88%	8%	3%	1%
000000000000001	**i+3**	88%	7%	3%	2%
000000000000001	**i+5**	85%	7%	4%	4%
000000000000001	**i+7**	85%	7%	5%	3%

Markov Weibull Model

A hybrid Markov Weibull model is used by AASHTOWare BrM to predict future condition state of bridge elements. The model constrains bridge deterioration to a maximum reduction of 1 condition state between each observation point. This ensures a smooth deterioration from Condition State 1 to Condition State 4 as would be expected of the observed inspection records. BrM's Markov-Weibull model uses the Weibull distribution for the transition from Condition State 1 to 2 and the Markov model for transitions between Condition State 2 to 3 and from 3 to 4. The Weibull function is used for the first transition period as it is able to more accurately reflect the delayed onset of deterioration than the Markov model can alone. The models are created using a transition probability matrix that describes the probability that an element will transition from its current condition state to a different condition state at its next inspection.

$$\begin{array}{c|cccc} & CS1 & CS2 & CS3 & CS4 \\ \hline CS1 & \multicolumn{4}{c}{} \\ \end{array}$$

$$\begin{array}{cc} \begin{array}{c} \\ CS1 \\ CS2 \\ CS3 \\ CS4 \end{array} & \begin{array}{c} \begin{array}{cccc} CS1 & CS2 & CS3 & CS4 \end{array} \\ \begin{bmatrix} P_{11} & P_{12} & 0 & 0 \\ 0 & P_{22} & P_{23} & 0 \\ 0 & 0 & P_{33} & P_{34} \\ 0 & 0 & 0 & 1 \end{bmatrix} \end{array} \end{array}$$

where,
P_{ii} is the probability an element remains in current condition state
$P_{ij} = 1 - P_{ii}$

Figure 5. Transition Probability Matrix

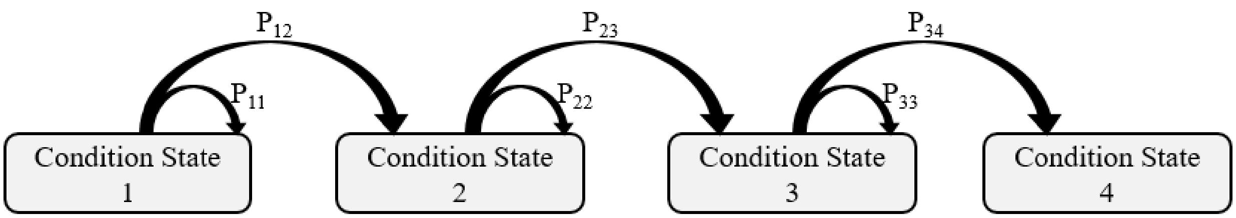

Figure 6. Element Deterioration

Using the deterioration model, 100% of an element will begin in Condition State 1 and gradually transition to Condition State 4 as the element ages. Figure 7 depicts deterioration using P11, P22, and P33 values as 0.966, 0.933, and 0.871, respectively. For each age along the horizontal axis, the percentage of an element in each of the four condition states can be read from the vertical axis.

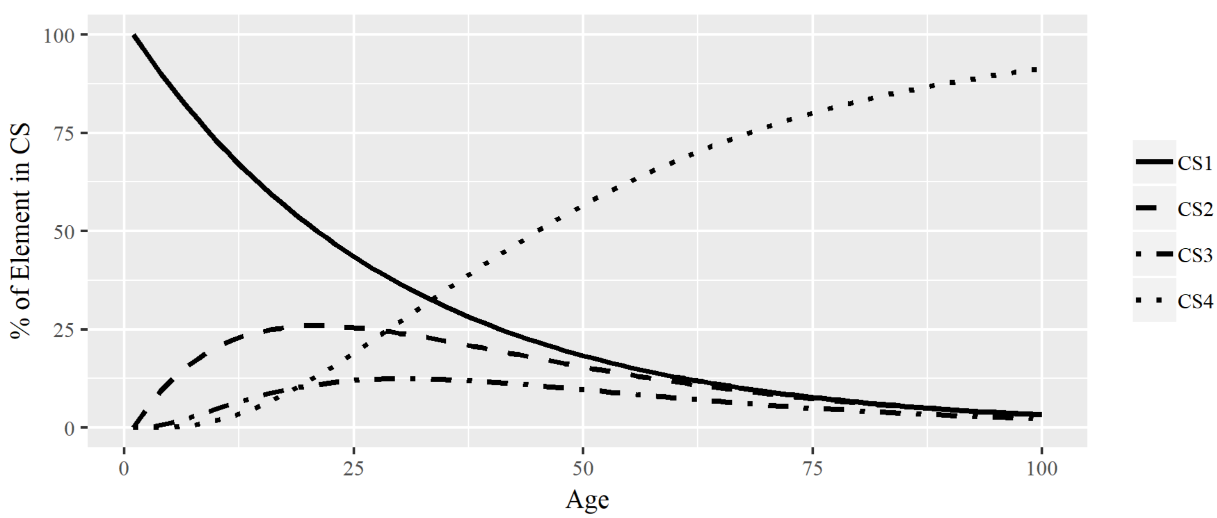

Figure 7. Example Deterioration Curve

BrM does not use transition probability distributions directly. Instead the probabilities must be converted into equivalent transition times, or the median number of years in which a unit of the element stays within its original condition state per Eqn. 3.

$P_{ii}=(0.5)^{1/T_{ii}}$, where [Eqn. 3], (AASHTO,2014 Eqn. 2)

P_{ii} is the probability an element remains in current condition state

T_{ii} is the time required for half of an element to transition out of current condition state, in years

Using Eqn. 3 , the deterioration probabilities used in Figure 7 translate to transition times me of 20 years, 10 years, and 5 years, respectively.

Model Optimization and Determination of Modification Factors

After inspection records were filtered and identified as belonging to a unique environment, tables were created by calculating the total quantity of elements in each condition state using the bridges age at time of inspection as the index (see Table 4). Additional fields were then created to calculate the total percentage of an element present in each condition state.

Table 4. Observed Condition States

	Element Quantity (ft, ft^2, etc)					% of Elements in CS			
Age	**CS1**	**CS2**	**CS3**	**CS4**	**Σ**	**CS1**	**CS2**	**CS3**	**CS4**
0	123983.5	0.0	0.0	0.0	123983.5	100%	0%	0%	0%
1	164661.4	0.0	0.0	0.0	164661.4	100%	0%	0%	0%
2	121930.8	12542.8	0.0	0.0	134473.6	90.7%	9.3%	0%	0%
.	.	.	.	.		.	.	.	.
.	.	.	.	.		.	.	.	.
.	.	.	.	.		.	.	.	.
50	114766.7	157166.7	91218.5	3716.8	366868.7	31.3%	42.8%	24.9%	1.0%

Model optimization for each study was completed as a two-step process using Microsoft Excel. First by optimizing the baseline model and then by optimizing each subsequent grouping. The data used as the baseline model was selected uniquely for each environment, but in general was selected as being the subset least impacted by the environment being studied. For instance, when investigating the impact of truck traffic the baseline dataset was selected as bridges exposed to a low levels of truck traffic. Bridges exposed to a high levels of truck traffic were assumed to be in a harsher environment and therefore would deteriorate at an accelerated rate.

The baseline model was optimized using the Excel Solver tool. During this step, transition times (T_{11}, T_{22}, T33) and the shape parameter, β, were optimized according to the following constraints. Transition times were restricted between 0.1 years and 500 years, and β between 1 and 10. These constraints help to prevent the model from solving to unrealistic deterioration rates. The optimization function was set to minimize the sum of squared errors between the observed percentage in each condition state and the modeled percentage in the same condition state for bridges aged 0 to 50 years old.

$$Error = \sum_{i=0}^{50}\sum_{j=1}^{4}(\%CS\ j\ _{Age=\ i}^{Observed} - \ \%CS\ j\ _{Age=\ i}^{Modeled})^2 \qquad \text{[Eqn. 4]}$$

Table 5 uses data from a Truck Traffic study of Element 12 for illustrative purposes.

Table 5. Model Optimization

	Observed				Model				Squared Error			
Age	**% in CS1**	**% in CS2**	**% in CS3**	**% in CS4**	**% in CS1**	**% in CS2**	**% in CS3**	**% in CS4**	**CS1**	**CS2**	**CS3**	**CS4**
0	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	100.0	0.0	0.0	0.0	98.2	1.8	0.0	0.0	3.3	3.3	0.0	0.0
2	90.7	9.3	0.0	0.0	96.2	3.8	0.0	0.0	30.4	30.5	0.0	0.0
.	.	.	.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.	.	.	.
50	29.6	58.8	11.5	0.0	28.4	57.9	12.6	1.1	1.6	1.0	1.3	1.2
										Total Error		**72.6**

Optimization of each subsequent group was completed by identifying a single modification factor, F, which can be multiplied by the previously determined transition times to best fit the data observed in the harsh environment as shown in Eqn. 5.

$$\mathrm{T}_{\mathrm{ii}}^{\mathrm{Environment}} = \mathrm{T}_{\mathrm{ii}}^{\mathrm{Baseline}} \times \mathrm{F} \quad \text{, where} \qquad \text{[Eqn. 5]}$$

$\mathrm{T}_{\mathrm{ii}}^{\mathrm{Environment}}$ are the transition times for an element in a harsh environment

$\mathrm{T}_{\mathrm{ii}}^{\mathrm{Baseline}}$ are the transition times for the baseline model

F is the environment modification factor

The modification factor was permitted to range between 0 and 99 and the model error (Eqn. 4) was again minimized using Microsoft Excel. For compatibility with the BrM software, the Weibull shape parameter, β, is not permitted to change between the baseline and any subsequent models. As shown in Table 6, a high presence of truck traffic resulted in a modification factor of 0.85 which reduces each transition time but has no effect on β.

Table 6. Application of Modification Factor on Element 12

	Low Truck Traffic	**High Truck Traffic**
β	1.08	1.08
T_1	28.77	24.54
T_2	90.62	77.31
T_3	143.76	122.65
Error	9768	66090
F	-	**0.85**

Modification factors less than 1.00 imply that transition times are reduced for elements in this environment, or that deterioration occurs quicker than the baseline model. Modification factors greater than 1.00 imply that transition times are longer and deterioration occurs slower than the baseline model.

Environment Factors

Eleven potentially relevant environment factors were investigated during this study at the request of VDOT. The environments identified do not attempt to be an exhaustive listing of environments in Virginia but rather a representative sample formed by prior VDOT experience. An explanation of each environment, the logic used to isolate the data, the quantity of bridges in each category, and a listing of key bridge elements that are expected to be directly impacted by the environment are detailed below.

District Maintenance Practices

The Virginia Department of Transportation is divided into nine districts as shown in Table 7. Each district is responsible for the construction and maintenance of bridges that fall within their jurisdiction. Due to this separation there may exist some disparity in maintenance or construction practices between each district. Additionally, Virginia's geographic diversity means each district is exposed to unique climates ranging from the eastern coast to the Blue Ridge Mountains. To explore these differences each districts deterioration rates were determined and compared to a baseline model created using state-wide data from all nine districts.

Many bridge elements may be affected by a bridges district classifications. Key elements such as concrete decks (element 12), concrete abutments (element 215), concrete pier caps (element 234), and steel girders (element 12) are widely abundant in all nine districts and can be used to gauge the effectiveness of district as a global environment factor

Table 7. District Groupings

District	Name
1	Bristol
2	Salem
3	Lynchburg
4	Richmond
5	Hampton Roads
6	Fredericksburg
7	Culpeper
8	Staunton
9	Northern Virginia

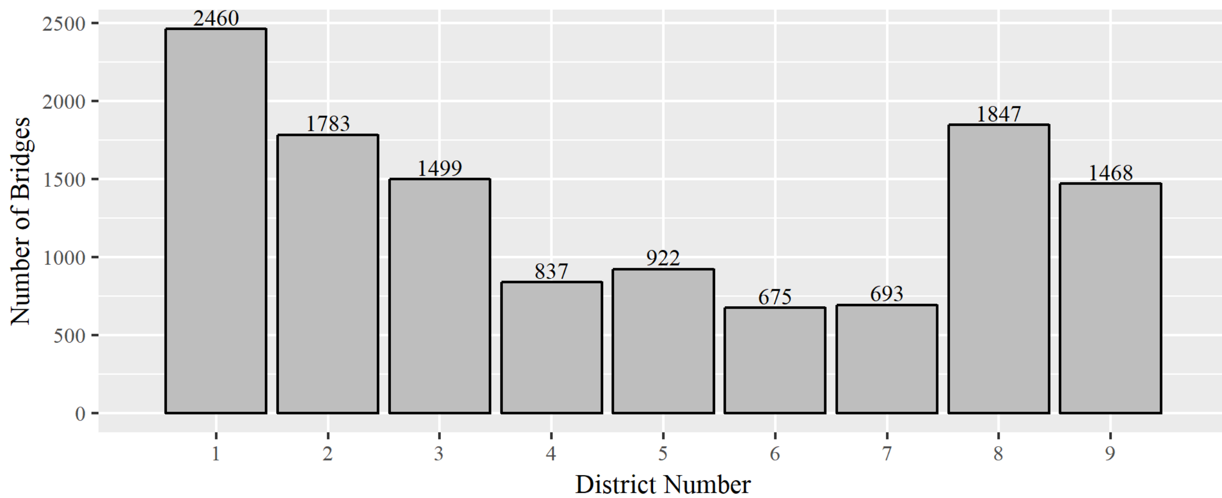

Figure 8. District Practices - Available Data

Functional Class

FHWA Federal Item 26 defines the functional classification of an inventory route carried by a bridge using Table 8 below.

Table 8. Functional Classifications

	Code	Description	Environment Grouping
Rural	01	Principal Arterial - Interstate	1
	02	Principal Arterial – Other	1
	06	Minor Arterial	1
	07	Major Collector	2
	08	Minor Collector	2
	09	Local	3
Urban	11	Principal Arterial – Interstate	1
	12	Principal Arterial – Other Freeways or Expressways	1
	14	Other Principal Arterial	1
	16	Minor Arterial	2
	17	Collector	2
	19	Local	3

Intuitively, bridges defined as arterial, collector, and local may be expected to deteriorate at different rates. Arterial and collector routes will have higher average daily traffic, truck traffic, and speed limits when compared to local routes. The increased loads are likely to cause quicker deterioration of bridge decks. Alternatively, arterial and collector routes may have a higher level of importance to the highway system as a whole when compared to local routes. Because of this, allocation of funds for minor repairs and general maintenance may not go towards local routes as frequently resulting in faster deterioration. To complete this study, bridges were classified into three environment groupings per Table 8. Group 1 bridges were used as the baseline model for this study.

With traffic loading and frequency being a primary difference between functional classifications, concrete decks (element 12) and steel girders (element 107) are key elements that may deteriorate quicker for arterial routes. Similarly, concrete abutments (element 12) and concrete pier caps (element 234) will experience heavy loading due to the functional classification and may see increased deterioration rates.

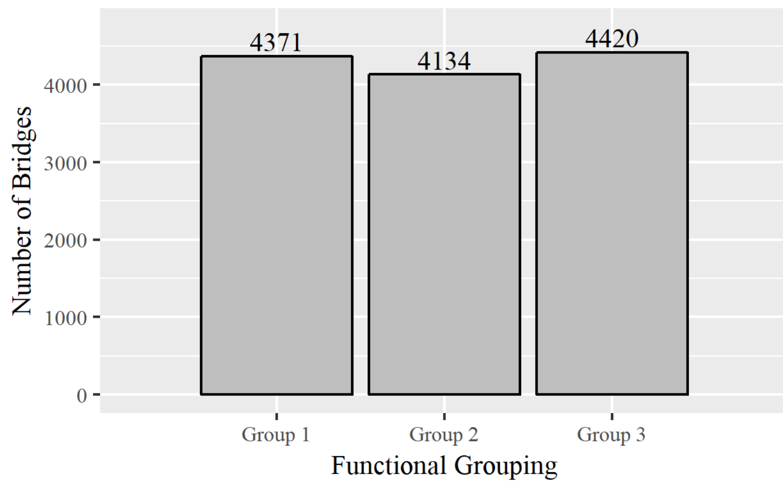

Figure 9. Functional Class – Available Data

Truck Traffic

Heavy vehicles such as 18-wheelers are known to cause significantly more damage to bridges and roadways than typical non-commercial vehicles (TRB, 2002). Federal Item 109 records the average daily truck traffic (ADTT) as a percentage of Federal Item 29, average daily traffic (ADT). To further capture the influence of truck traffic the number of truck lanes was also considered for this study. Various state laws apply lane restrictions to truck traffic, often prohibiting trucks from using the far left lane (Code of Virginia, 2004). To account for this reduced lane availability, Federal Item 102 and 28A were also used. Item 28A reports the number of lanes on the structure and 102 reports the direction of traffic, i.e., 1-way traffic, 2-way traffic, or a 1-lane bridge. Table 9 was created to predict the number of truck lanes based on this information.

Table 9. Truck Lanes

	Number of Truck Lanes		
Lanes	1-Lane Bridge	1-Way Traffic	2-Way Traffic
1	1	1	-
2	-	1	2
3	-	2	2
4	-	2	2
5	-	2	3
6+	-	2	4

Average daily truck traffic per truck lane was than determined using Eqn. 6.

$$\text{ADTT}= \frac{\text{ADT}}{\text{Truck Percent}} \div \text{Truck Lanes} \qquad \text{[Eqn. 6]}$$

High ADTT was defined as bridges having more than 3000 trucks per truck lane per day. Low ADTT was used as the baseline model. A sensitivity analysis of this threshold allowed for a reasonable subset of bridges to be categorized as high truck traffic.

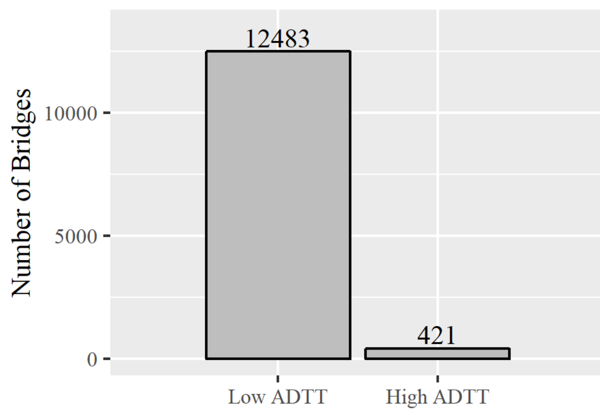

Figure 10. ADTT per Truck Lane - Available Data

Heavy volumes of truck traffic is expected to increase the deterioration of concrete decks (element 12) and concrete abutments (215) due to the constant impact and friction from truck tires. Similarly, the dynamic loading from heavy truck traffic is likely to increase the possibility of fatigue related damage to steel girders (element 107) (TRB, 2002).

Joint Presence

Multi-span bridges can be designed as either simply supported or continuously supported. As seen in Figure 11, a simply supported bridge (a) will introduce joints at each of the spans bearing points, whereas a continuously supported bridge (b) will only have joints at the abutments.

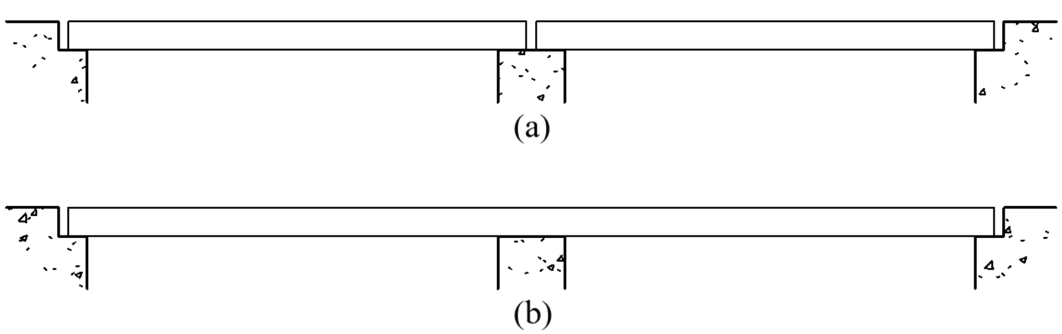

Figure 11. Bridge Geometry with (a) Joint (b) No Joint

The presence of joints on a bridge allows water to penetrate to the substructure and exacerbates the deterioration of bridge bearings and pier caps as well as other subordinate bridge elements. The local environment around a bridge joint is expected to be much harsher than a similar bridge that does not have joints.

The current VDOT bridge inspection database does not explicitly identify which bridges are in a local environment subjected to the presence of a joint. However, a method was established during this study to predict joint presence using bridge parameters that are recorded. Federal Items 52 (deck width), 34 (skew), and 45 (main spans) were used in conjunction with bridge joints coded as elements 300 to 306 to predict joint presence.

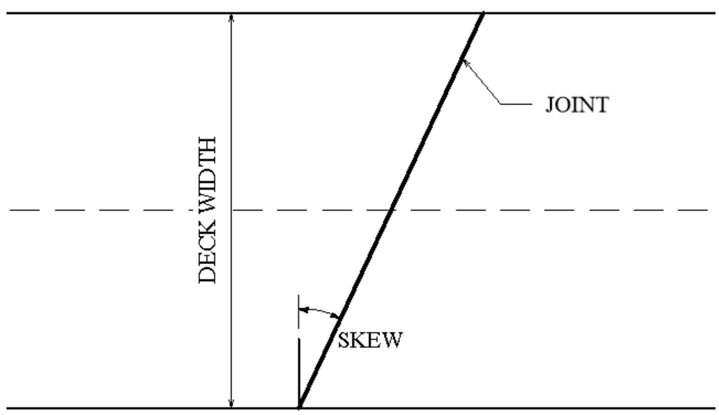

Figure 12. Joint Length

For a continuously supported bridge, joints are expected to be found exclusively at the beginning and end of the bridge and run the full out-to-out dimension of the bridge deck. Therefore it would be expected that the total length of joints could be predicted using Eqn. 7.

$$\sum(\text{Joint Lengths}) = \frac{\text{Deck Width}}{\cos(\text{Skew})} \times 2 \qquad \text{[Eqn. 7]}$$

For a simply supported bridge, joints would be found at the beginning and end of every span, running the full out-to-out dimension of the bridge deck. For instance, a 3 span bridge would have a joint at each of the abutments and also at both of the interior bearing points. Using this assumption, the total length of joints on a simply supported bridge can be predicted using Eqn. 8.

$$\sum(\text{Joint Lengths}) = \frac{Deck\ Width}{cos(Skew)} \times (Spans + 1) \qquad \text{[Eqn. 8]}$$

To allow for discrepancies in Eqn. 7 and Eqn. 8, a ± 20% allowance of the expected total joint length was permitted. Bridges that did not fall into one of the two categories were excluded from the study. Bridges with a joint present were used as the base model.

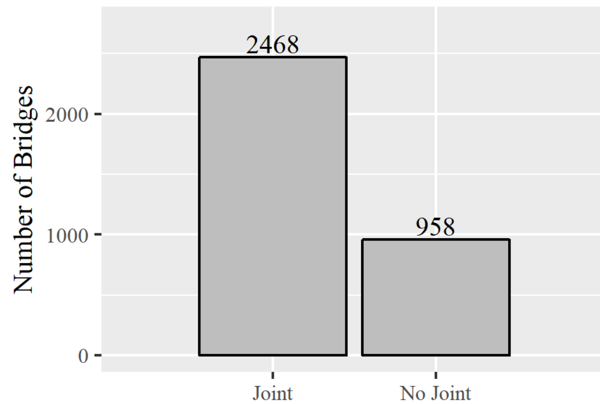

Figure 13. Joint Presence - Available Data

It is expected that the deterioration of bearing elements such as elastomeric bearings (element 310), moveable bearings (element 311), enclosed bearings (element 312), and fixed bearings (element 313) will be greatly impacted by the presence or absence of a joint. Concrete abutments (element 215) and concrete pier caps (element 234) are also expected to be impacted by joint presence due their proximity to joint locations on bridges. However, both bridges defined as having a joint and not having a joint will still have joints present at the abutment (Figure 11), because of this the results for element 215 may not be intuitive.

Lateral Splash Zone

In wet conditions vehicular traffic passing below a bridge can spray standing water back onto bridge elements nearest the traffic. The repeated splashing potentially creates a harsh wet environment that can last much longer than the original weather event that brought the water. While the bridge management database records the minimum right and left horizontal clearances below the bridge, Federal Items 55 and 56 respectively, the database does not explicitly indicate which bridge element is nearest the road way. Many bridges in Virginia do not have vehicular traffic passing beneath the structure and therefore would not be subjected to a splash zone. As such, all bridges defined as being in a splash zone must have Federal Item 28B greater than or equal to 1, indicating that the structure has at least one traffic lane passing beneath the structure.

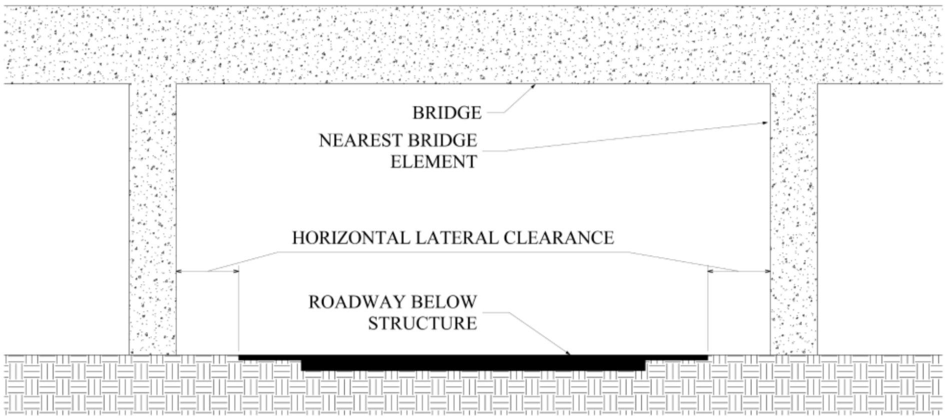

Figure 14. Lateral Splash Zone

Using these requirements, high and low clearances are defined in Table 10. The demarcation of low clearance was chosen to provide an adequate sample size (Figure 15) while still being small enough to reflect the described environment. Bridges with a high lateral clearance were used as the baseline model and are expected to deteriorate slower than those with a low lateral clearance.

Table 10. Lateral Splash Zone Groupings

Group	Minimum Lateral Clearance
High Clearance	> 4'-6"
Low Clearance	≤ 4'-6"

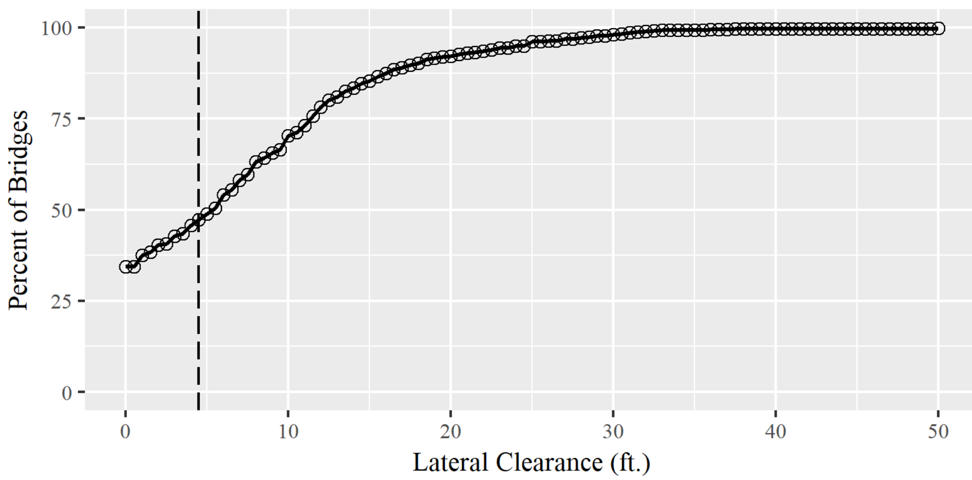

Figure 15. Lateral Clearance Distribution

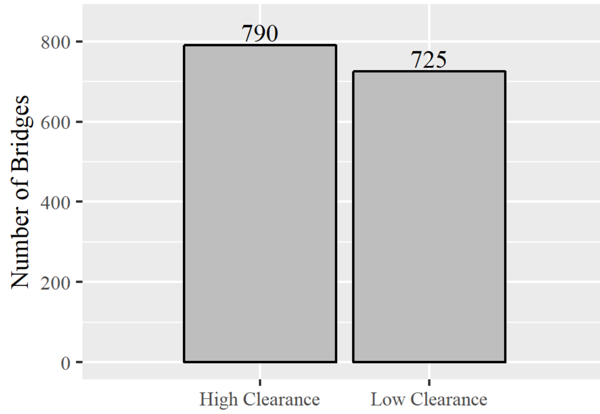

Figure 16. Lateral Splash Zone - Available Data

Lateral splash zones are expected to primarily impact substructure elements such as concrete columns (element 205), concrete pier walls (element 210), concrete abutments (element 215), and concrete pier caps (element 234) since they are the elements most likely to be splashed by traffic passing beneath the structure.

Vertical Splash Zone

Similarly to lateral splash zones, a bridge that causes low vertical clearance for traffic below may be subjected to splash or spray.

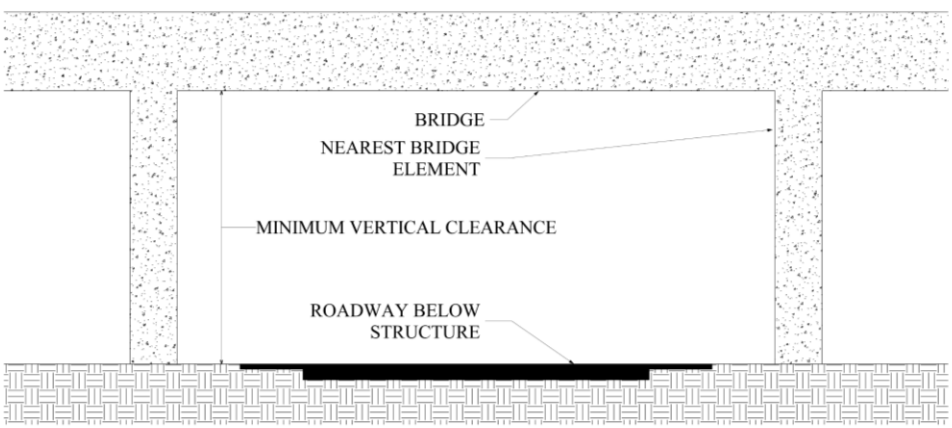

Figure 17. Vertical Splash Zone

Using the same Federal Item 28B requirements and Federal Item 54B (vertical clearance under bridge), bridges were classified as according to Table 11. The demarcation of low clearance was chosen to provide an adequate sample size (Figure 18) while still being small enough to reflect the described environment. Bridges with high vertical clearances were used as the baseline model and are expected to deteriorate slower than bridges with a low vertical clearance.

Table 11. Vertical Splash Zone Groupings

Group	Vertical Clearance
High Clearance	> 15'-0"
Low Clearance	≤ 15'-0"

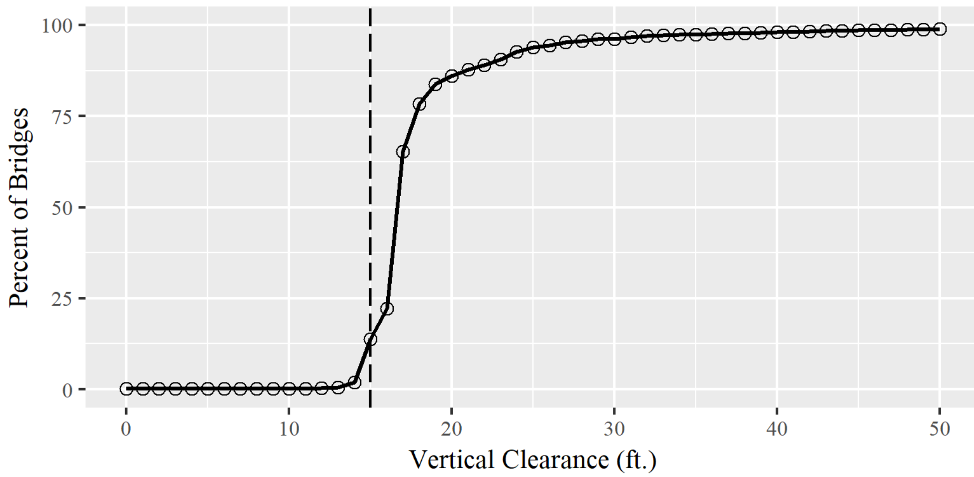

Figure 18. Vertical Clearance Distribution

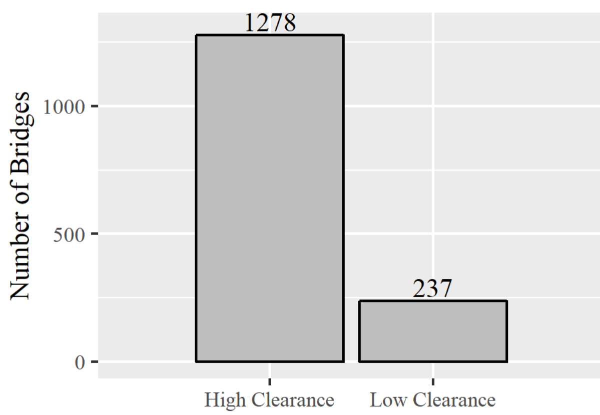

Figure 19. Vertical Splash Zone - Available Data

Similar to lateral splash zones, vertical splash zones are expected to primarily impact substructure elements. Concrete columns (element 205) and concrete pier caps (element 234) are expected to deteriorate quicker when a bridge has low vertical clearance. Superstructure elements such as steel beams (element 107) and concrete decks (element 12) may also be exposed to the effects of vertical splash zones.

Waterways

Many bridges in the state of Virginia span across waterways. State Item 42, the tidal indicator, is used to describe a bridges geometry relative to any waterways passing below the structure. State Item 42 is broken into three categories (1) bridge does not cross water, (2) bridge crosses water, or (3) bridge crosses water and has an element in the waterway. Federal Item 42B details the traffic type that is served under the bridge. Federal Item 42B coded as 5 indicates that the service type of the route beneath a bridge is a waterway. Using both state item 42 and Federal Item 42B the environment classifications were established as shown in Table 12. The base models were created using the data designated as Tide = 0 and are expected to deteriorate slower than those designated as Tide = 1 and 2.

Table 12. Waterways Groupings

Group	State Item 42	Federal Item 42B
Tide = 0	Not over water	-
Tide = 1	Over water	-
Tide = 2	Over water with element in water	(5) Waterways

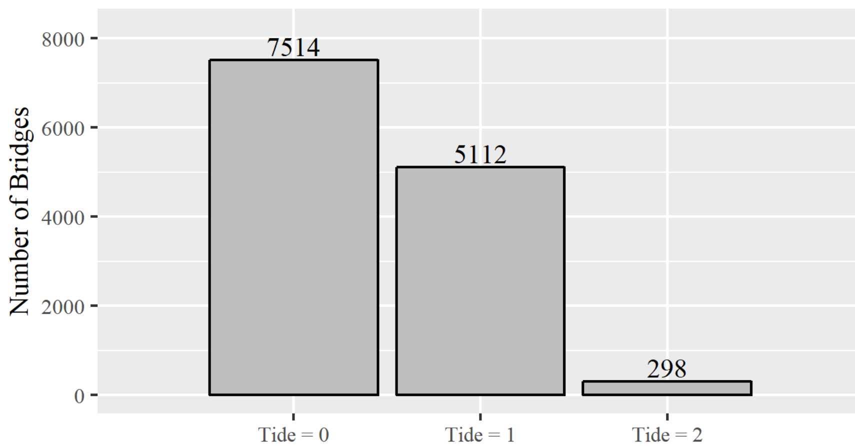

Figure 20. Waterways - Available Data

It is expected that bridge elements physically in water such as prestressed columns (element 204) and concrete columns (element 205) will deteriorate quicker than the baseline model. It is unlikely that concrete abutments (element 215) will be located within a waterway, however abutments may become submerged for extended periods of time during flooding events and will also be used as key indicator element.

Brackish Water

The Chesapeake Bay on Virginia's East coast is composed of brackish water that dissipates in salinity as it branches inland. To determine the effect that higher salinity waters have on bridge deteriorations it is needed to define salinity ranges within the Chesapeake Bay. A previous study obtained water samples from 16 various locations and measured the salinity levels in parts per trillion (ppt) (Whitehead, Roach, Zhang, & Glavez, 2011). These maps were reproduced using ArcMap as shown in Figure 21.

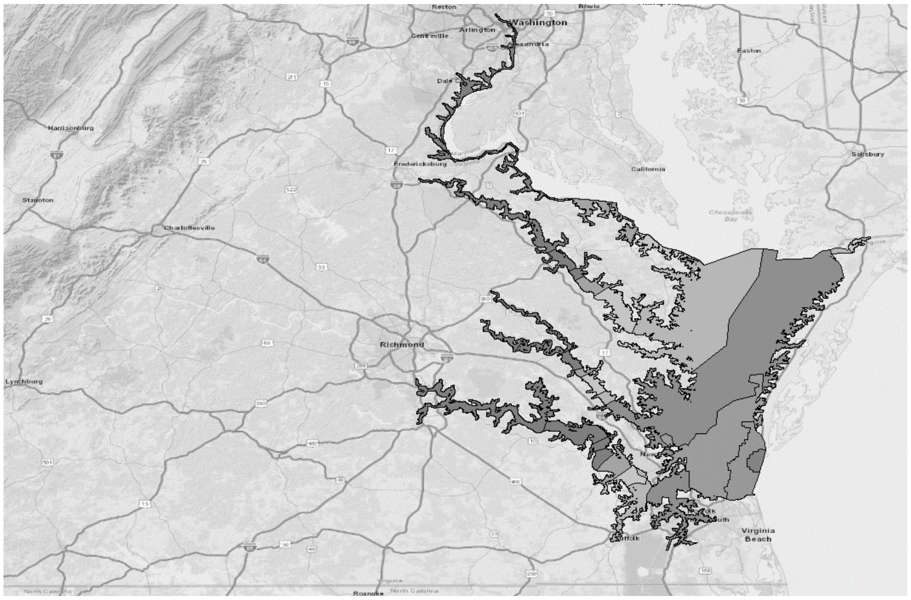

Figure 21. Chesapeake Bay Salinity Map

Similarly to the aforementioned Waterways study, Federal Item 42B and State Item 42 were used in conjunction with the salinity map to classify bridges into unique exposure categories (Table 13). This study is an extension of the waterways study and further splits the most severe environment into three separate categories, Tide = 2, 3, and 4.

Table 13. Brackish Water Groupings

Group	State Item 42	Federal Item 42B	Distance to Salinity Zone	Salinity (ppt)
Tide = 0	Not over water	-	-	-
Tide = 1	Over water	-	-	-
Tide = 2	Over water with element in water	(5) Waterways	> 0.5 Miles	-
Tide = 3	Over water with element in water	(5) Waterways	≤ 0.5 Miles	≤ 10
Tide = 4	Over water with element in water	(5) Waterways	≤ 0.5 Miles	> 10

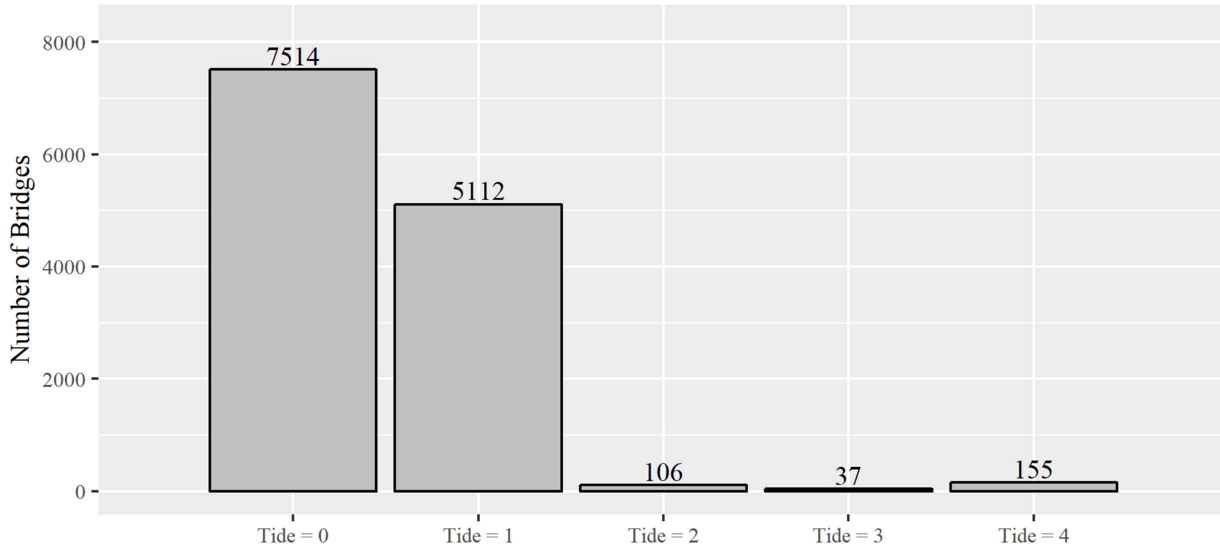

Figure 22. Brackish Water - Available Data

As this is an extension of the waterways study, prestressed columns (element 204), concrete columns (element 205), and concrete abutments (element 215) will again be used as key elements to gauge the quality of the environment classification.

Freeze-Thaw Cycles

The expansion and contraction of water during freeze-thaw cycles is expected to accelerate deterioration of concrete bridge elements. The National Solar Radiation Data Base has published a data set called TMY3, which contains a full year of hourly meteorological data (or a typical meteorological year). This data was created from multiple years' worth of observations from over 1000 locations across the United States and its territories (Sengupta, et al., 2018). The data is intended to represent the weather that would occur throughout a typical year and is often used for computer simulations relating to solar energy and sustainability calculations.

Amongst the data provided in the TMY3 database are records of the predicted hourly temperature. Using the temperature data, the number of times a station underwent a freeze-thaw cycle could be tabulated. A freeze-thaw cycle was defined as going from above freezing, to below freezing, and then back above freezing for any length of time. The quantity of freeze-thaw cycles were overlaid on a map and regions were interpolated using the Kriging method using GIS to determine the expected number of freeze-thaw cycles that each bridge in the database is likely to experience. A bridges global environment was defined as having either low or high freeze-thaw counts per Table 14. Bridges with a low number of freeze-thaw cycles were used as the baseline model and are expected to deteriorate slower than those with a high number of freeze-thaw cycles.

Table 14. Freeze-Thaw Cycles Groupings

Group	Freeze-Thaw Cycles
Low	< 78
High	≥ 78

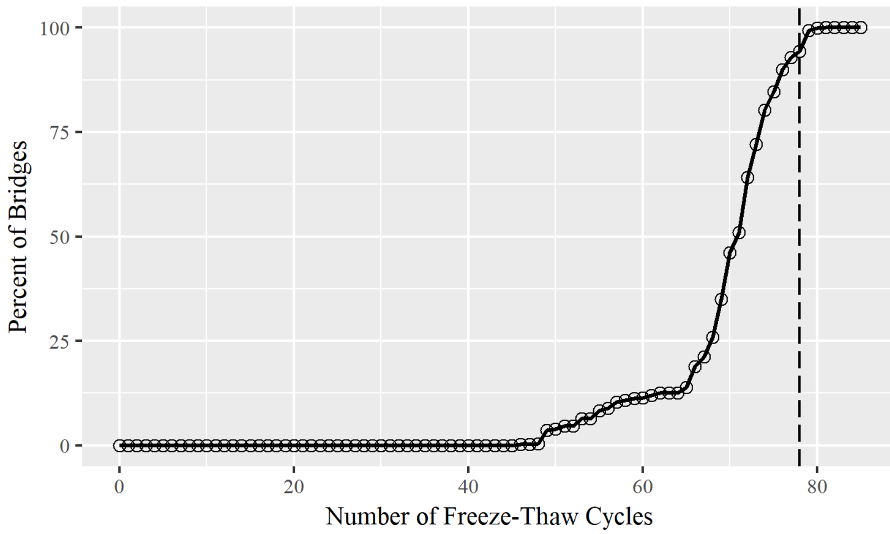

Figure 23. Freeze-Thaw Cycles Distribution

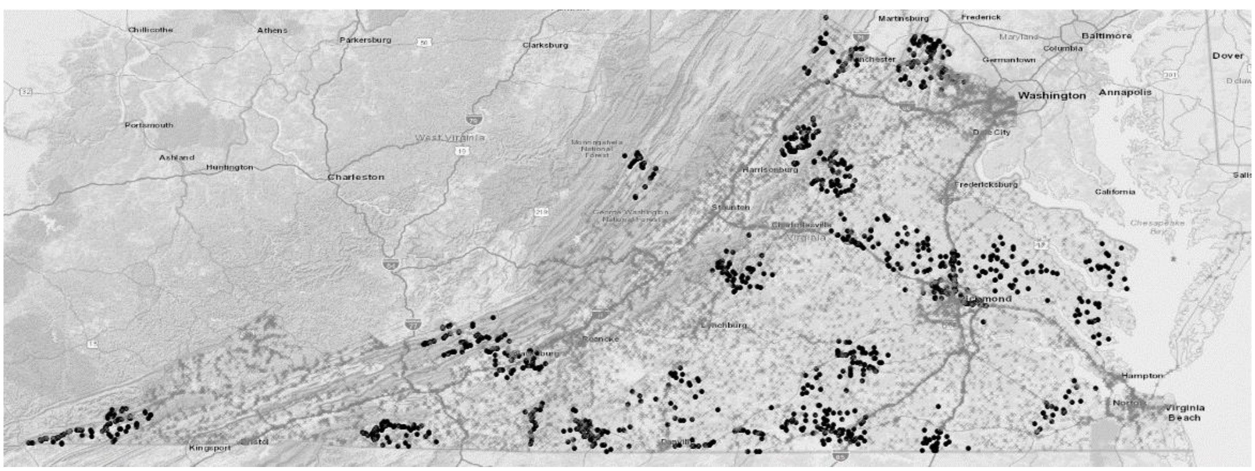

Figure 24. Freeze-Thaw Regions

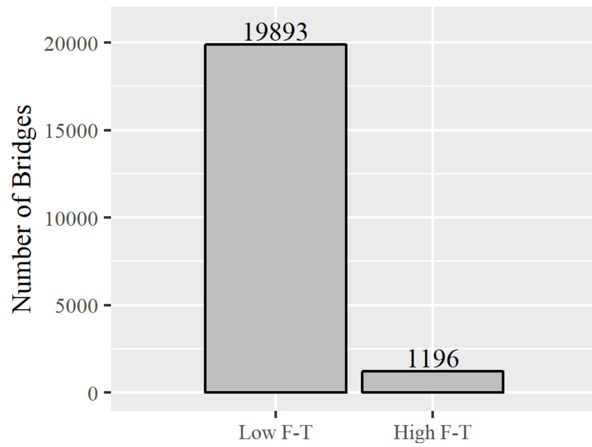

Figure 25. Freeze-Thaw - Available Data

Concrete bridge elements subjected to the expansion and contraction of freezing water are the primary interest in this study. Concrete bridge decks (element 12), columns (element 205), abutments (element 215), and pier caps (element 234) are all expected to deteriorate quicker when exposed to an increased number of freeze-thaw cycles.

Temperature Extremes

Using temperature values in the TMY3 database, the temperature range a bridge is exposed to throughout a year can determined by subtracting the maximum and minimum temperatures observed at each weather station. These values were then used to create regions in GIS to associate a temperature range for each bridge in the database.

Table 15 was used to classify the global environment as having high or low temperature ranges. Bridges with a low temperature range were used as the baseline model and are expected to be in a mild environment therefore deteriorating at a slower rate.

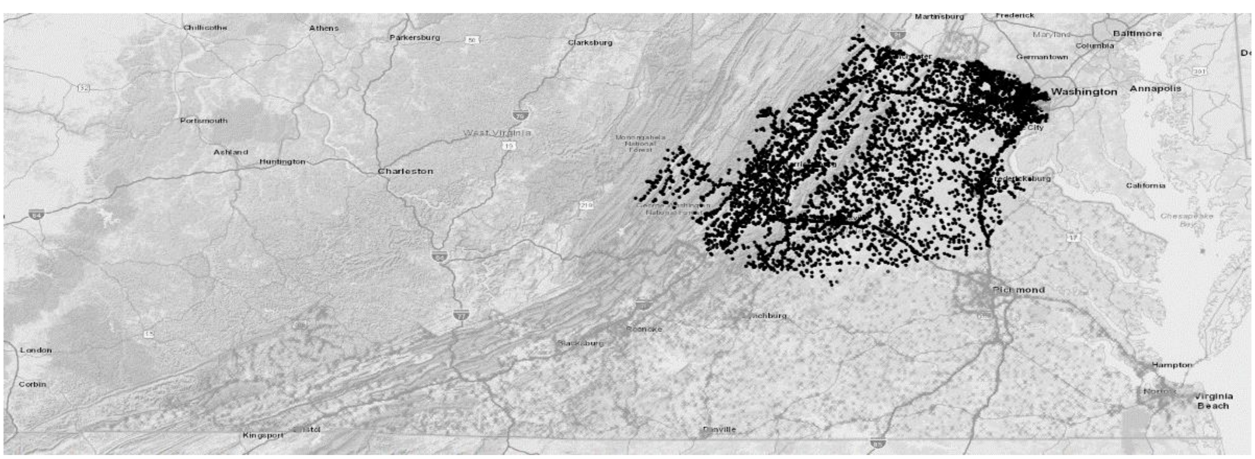

Figure 26. High Temperature Range

Table 15. Temperature Range Groupings

Group	Temperature Range
Low	< 98°F
High	≥ 98°F

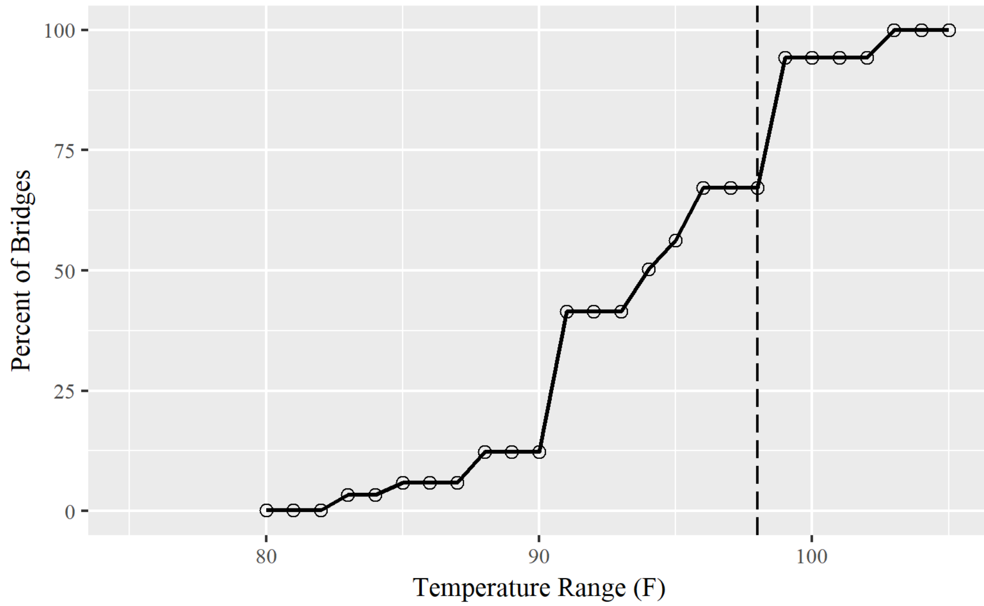

Figure 27. Temperature Range Distribution

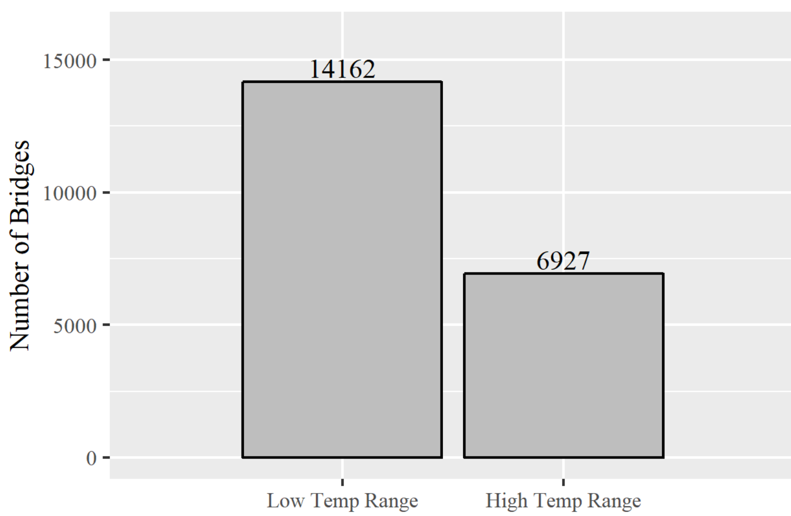

Figure 28. Temperature Range - Available Data

It is expected that bridges exposed to a high temperature ranges annual may be more prone to deterioration as the expansion and contraction of elements may weaken particular elements. Concrete bridge decks (element 12), columns (element 205), abutments (element 215), and pier caps (element 234) are all expected to deteriorate quicker when exposed to a high temperature range throughout the year.

Coastal Areas

Bridges located near the coastline are likely subjected to a harsher global environment due to the presence of airborne chlorides. The additional salts in the air may result in quicker deterioration of both concrete and steel bridge elements. Using a proximity analysis in ArcMap the distance between each bridge and the nearest point to the Eastern coast line was calculated. Bridges further than 10 miles away from the coast were used as the baseline model and those defined as “Coastal” are expected to deteriorate quicker.

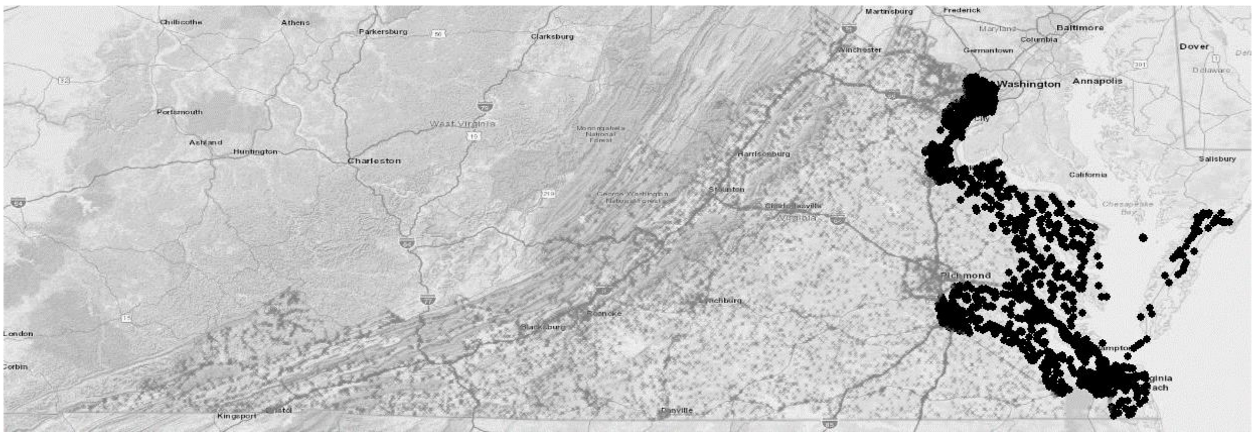

Figure 29. Coastal Regions

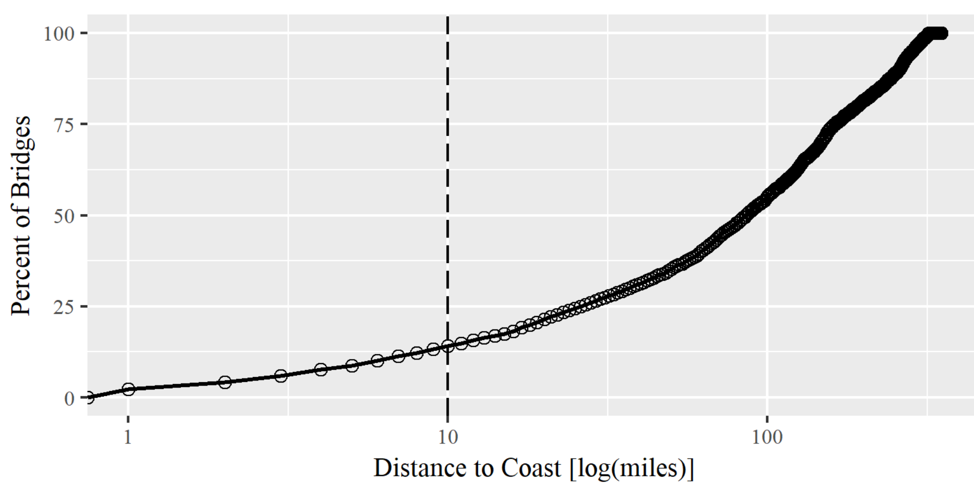

Figure 30. Coastal Distance Distribution

Table 16. Coastal Areas Groupings

Group	Distance to Coast
Not Coastal	> 10 Miles
Coastal	≤ 10 Miles

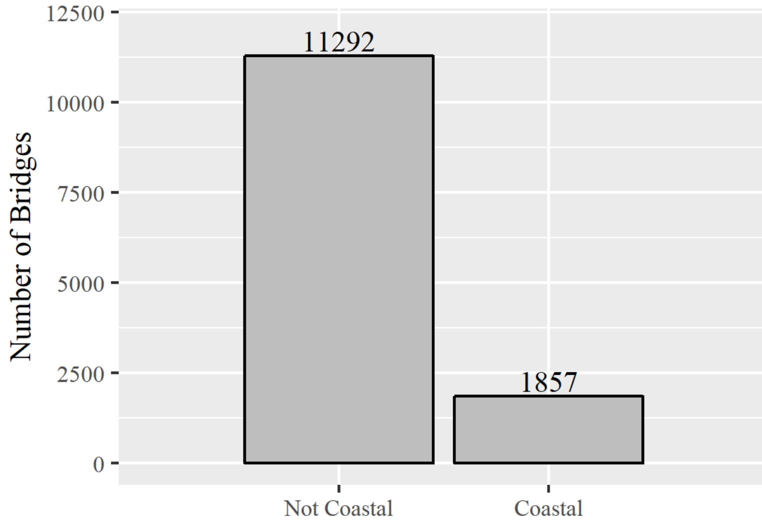

Figure 31. Coastal - Available Data

Many bridge elements may be impacted by the proximity to the coast. As with several other environment studies, concrete decks (element 12), steel girders (element 107), concrete columns (element 205), concrete abutments (element 215), and concrete pier caps (element 234) have been identified as key elements to help determine the impact of this environment.

Combined Modification Effects

Individual bridges are not unique to a single environment but rather are subjected to multiple environments at a time. An effective way to combine environmental factors needs to be determined as well as concluding which data sample should be used as the baseline model. For the purposes of this investigation, state-wide deterioration rates of a given element were used as the baseline model. Unique environmental factors were combined multiplicatively to develop an effective environmental modification factor using Eqn. 9.

$$f^{\text{E}} = f^{\text{District}} * f^{\text{Functional Class}} * \ldots * f^{\text{Coastal}}, \text{ where} \qquad \text{[Eqn. 9]}$$

f^{E} is the effective environmental adjustment factor

$f^{\underline{\text{xx}}}$ are the adjustment factor for a given environment

The eleven environments can be combined in 2047 unique combinations ($\sum_{K=1}^{11}\binom{11}{K}$). Due to this large number, it is impractical to determining the ideal combination of environments that would result in a deterioration curve most representative of each unique bridge. Successful implementation of environment factors into BrM will result in subsets of bridges being more accurately predicted than what is currently possible using only the state-wide deterioration parameters. Various combinations of environments are presented in the Results and Discussion portion of this report.

RESULTS AND DISCUSSION

Low Level Maintenance

Comparison of improvement filters

Table 17 provides an example of the quantity of inspection records that are removed when using each improvement filter method on a dataset consisting of inspection records of concrete abutments (element 215).

Table 17. Improvement Filters on Element 215 Data

Method[1]	LLM[2]	Original	Age Removed	Filter Removed	Remaining Records	% Removed
1	-	61569	214	0	61355	0.35%
2	-	61569	214	29682	31673	48.56%
3(a)	1%	61569	214	27223	34132	44.56%
3(b)	3%	61569	214	22866	38489	37.49%
3(c)	5%	61569	214	19668	41687	32.29%
4(a)	1%	61569	214	28762	32593	47.06%
4(b)	3%	61569	214	23902	37453	39.17%
4(c)	5%	61569	214	20550	40805	33.72%

[1] Refer to Table 1 for full description of each improvement filter method
[2] LLM or Low Level Maintenance is the percentage of improvements allowed between inspection events, refer to Table 1 for further detail

Method 2 is the harshest filter and removes inspection records with any improvement in Condition States 1 and 4, as a result 48.56% of data was removed for element 215. Methods 3(c) and 4(c) are the most lenient on movement between condition states removing 32.29% and 33.72% of data, respectively. It was noted that the example above removes a significant amount of data from the available database, however fewer inspection records were removed in identical studies using different bridge elements.

Impact of improvement filters on overall deterioration rates

Table 18 shows the differences in deterioration parameters due to selection of improvement filter for reinforced concrete abutments (element 215).

Table 18. Variation in Deterioration Parameters Due to Improvement Filter

	Method							
	1	**2**	**3(a)**	**3(b)**	**3(c)**	**4(a)**	**4(b)**	**4(c)**
T11	150.38	105.15	110.15	121.03	129.54	121.38	128.78	135.95
Beta	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
T22	50.26	54.78	53.55	52.91	52.81	50.13	51.50	52.09
T33	211.86	211.82	196.68	182.67	200.62	393.04	373.61	427.42
∑ Error	98.29	363.94	291.28	195.85	161.83	200.80	158.98	147.59

Without removing any inspection records (method 1) , the model predicts a transition time from Condition State 1 to Condition State 2 of 150.38 years (T11) whereas the harshest filter (method 2) predicts 105.15 years to transition. Recalling that the transition times (T11, T22, and T33) are defined as "the median number of years in which a unit of the element stays within its original condition state". In other words, method 1 predicts that after 150.38 years, only 50% of the element quantity will remain in Condition State 1. T22 was not found to be greatly affected by selected improvement filter. Methods 1,2, and 3(a-c) all predicted similar T33 transition times, while methods 4(a-c) predicted elements stay in Condition State 3 for a longer period of time before transitioning to Condition State 4. This is not unexpected as methods 4(a-c) are the only methods that account for the cumulative percentages of elements in multiple condition states at a time (see Table 1).

Through these findings and expert elicitation with the projects advisory panel, it was decided to conduct the environmental studies using method 4(b). Method 4(b) strikes a balance of permissible inspector variability between Condition State 1, 2, and 3 while maintaining a reasonable amount of data for the studies to still be considered representative of the entire database.

Environment Effects

A wide range of bridge elements were analyzed for each environmental study and comprehensive tables of results are provided in Appendix B. For each study an abbreviated table of results is provided in this section. Modification factors less than 1.00 imply that transition times are reduced for elements in this environment, or that deterioration occurs quicker than the baseline model. Modification factors greater than 1.00 imply that transition times are longer and deterioration occurs slower than the baseline model. Of the results, many of elements exhibit the behavior expected in a harsh environment, however some elements respond inversely to the expected behavior. Additionally, some element models predict 500 year transition times for an element to move from Condition State 3 to Condition State 4.

District Practices

Table 19. Key Elements - District

Element		District								
Number	Name	All Districts				1	2	3	4	5
		T11	Beta	T22	T33	F	F	F	F	F
12	Concrete Deck	22.17	1.06	71.26	22.46	0.97	1.53	1.28	1.94	0.79
107	Steel Girder	44.64	1.23	25.12	44.60	0.75	0.81	1.79	0.76	1.14
215	Concrete Abutment	126.25	1.00	49.69	174.97	0.86	1.41	1.28	0.52	0.81
234	Concrete Pier Cap	67.64	2.02	7.95	500.00	0.87	0.79	1.49	0.95	1.25

Table 20. Key Elements - District (continued)

Element		District							
Number	Name	All Districts				6	7	8	9
		T11	Beta	T22	T33	F	F	F	F
12	Concrete Deck	22.17	1.06	71.26	22.46	0.96	1.66	1.25	0.77
107	Steel Girder	44.64	1.23	25.12	44.60	0.85	0.78	0.55	1.63
215	Concrete Abutment	126.25	1.00	49.69	174.97	0.91	0.80	1.48	0.97
234	Concrete Pier Cap	67.64	2.02	7.95	500.00	0.96	1.05	0.99	0.86

It was anticipated that classifying element inspection records by VDOT district could reveal an overall trend showing that bridges in particular districts may deteriorate at different rates than the overall state-wide average. Using the key elements in Table 19 and Table 20 this was shown to be true of district 1, 3, and 6. Elements in districts 1 and 6 were shown to deteriorate at a faster rate than the state-wide model. This is an interesting finding as district 1 and district 6 are located on opposite sides of the state and are unlikely to share multiple similarities in terms of additional global environment classifications. District 3 elements were found to deteriorate at a slower rate than the state-wide model potentially indicating that the district has more effective maintenance practices or that its environment as a whole is less harmful on bridge elements.

Functional Class

Table 21. Key Elements - Functional Class

Element		Functional Classification					
Number	Name	Arterial (1, 2, 6, 11, 12, 14)				Collector (7, 8, 16, 17)	Local (9, 19)
		T11	Beta	T22	T33	F	F
12	Concrete Deck	19.77	1.27	113.74	43.85	1.27	1.95
107	Steel Girder	63.60	1.08	34.46	60.83	0.77	0.64
215	Concrete Abutment	116.34	1.00	35.86	266.31	1.27	1.13
234	Concrete Pier Cap	60.72	2.25	5.82	500.00	1.16	1.50

Concrete decks are shown to deteriorate quickest when located on an arterial bridge and condition state transition times were modeled to be 1.27 time slower for collector routes and a significant 1.95 times slower for local routes. This pattern was also seen for concrete pier caps. Concrete abutments deterorate quickest for arterial routes, but interestingly local routes were shown to be a harsher environment than collector routes. Steel girders were shown to have the opposite behavior and deteriorated faster on collector routes (F=0.77) and local routes (F=0.64).

Truck Traffic

Table 22. Key Elements - Truck Traffic

Element		Truck Traffic				
Number	Name	Low				High
		T11	Beta	T22	T33	F
12	Concrete Deck	28.77	1.08	90.62	143.76	0.85
107	Steel Girder	62.42	1.15	34.07	82.76	0.64
215	Concrete Abutment	147.25	1.00	56.58	266.21	1.07

Higher volumes of truck traffic resulted in a faster deterioration of both concrete decks (F=0.85) and steel girders (F=0.64), these results are consistent with the expected behavior. Concrete abutments were modeled to deteriorate at a slightly slower rate when exposed to high truck traffic (F=1.07). This is opposite of the expected behavior, however a modification factor of 1.07 only has a moderate effect on transition times. For example the median transition time from Condition State 1 to 2 (T11) was modeled to be 157.56 years as opposed to 147.25 years.

Joint Presence

Table 23. Key Elements - Joint Presence

Element		Joint Presence				
Number	Name	Joint				No Joint
		T11	Beta	T22	T33	F
215	Concrete Abutment	152.11	1.00	45.97	298.81	0.92
234	Concrete Pier Cap	67.39	2.04	8.04	500.00	1.11
310	Elastomeric Bearing	45.33	1.36	280.29	27.38	1.65
311	Moveable Bearing	29.45	1.80	157.61	51.01	1.23
312	Enclosed Bearing	53.90	10.00	11.32	500.00	1.00
313	Fixed Bearing	32.73	1.56	218.84	31.06	1.33

Consultation with personnel experienced in the industry suggest that the presence of a joint will result in significantly faster deterioration rates for elements located near the joint. This assumption was validated by the models for each of the bearing elements, most notably for elastomeric bearings (F=0.65). Enclosed bearings were found to not be affected by the presence or absense of a bridge joint (F=1.00). Concrete abutments were modeled to deteriorate faster in the absense of a joint – a result that is opposite of the expected conclusion. Revisiting Figure 11, both classifications in this study expect a joint to be present at the start and end of a bridge. Design standards for the width of joints in a continuous bridge compared to a simply supported bridge could be a possible explination of the models findings.

Lateral Splash Zone

Table 24. Key Elements - Lateral Splash Zone

Element		Lateral Splash Zone				
Number	Name	High Clearance				Low Clearance
		T11	Beta	T22	T33	F
205	Concrete Column	56.47	2.53	7.09	500.00	0.95
210	Concrete Pier Wall	60.69	2.46	8.38	500.00	1.02
215	Concrete Abutment	108.12	1.00	41.84	87.01	1.14
234	Concrete Pier Cap	74.73	1.53	9.12	500.00	0.91

Lateral splash zones were shown to have minimal effects on concrete columns and pier walls. Concrete pier caps exhibited the expected behavior and were shown to deteriorate faster (F=0.91) in a lateral splash zone. Concrete abutments however deteriorate slower (F=1.14) when in the presence of a splash zone.

Vertical Splash Zone

Table 25. Key Elements - Vertical Splash Zone

Element		Vertical Splash Zone				
Number	Name	High Clearance				Low Clearance
		T11	Beta	T22	T33	F
12	Concrete Deck	23.84	1.32	54.76	161.30	1.25
107	Steel Girder	58.04	1.00	30.36	53.83	1.47
205	Concrete Column	56.10	2.48	6.06	500.00	1.02
234	Concrete Pier Cap	66.52	1.81	7.43	500.00	0.82

Similarly to lateral splash zones, a vertical splash zone was shown to have little impact on concrete columns, the combiniation of these two findings may suggest that water sprayed on concrete columns runs down a column quick enough to not have significant impact on deterioration rates. In agreement with the previous study, a vertical splash zone was shown to be a harsh environment for concrete pier caps (F=0.82). Contrary to the initial assumption, the models show that concrete decks and steel girders deteriorate slower when in the presence of a vertical splash zone. Other unstudied environment factors are likely contributing to these findings.

Waterways

Table 26. Key Elements - Waterways

Element		Waterways					
Number	Name	Tide = 0				Tide = 1	Tide = 2
		T11	Beta	T22	T33	F	F
204	Prestressed Concrete Column	105.73	1.00	24.39	156.59	0.57	0.74
205	Concrete Column	59.87	2.27	6.40	500.00	0.94	1.36
215	Concrete Abutment	150.61	1.00	56.59	307.48	0.95	0.81

It was expected that Tide = 0, Tide = 1, and Tide = 2 would be sequentially harsher environments and result in faster deterioration rates accordingly. This behavior was seen for concrete abutments but not for prestressed or traditionally reinforced concrete columns. Prestressed concrete columns located in a water way (Tide = 2) deteriorated slightly slower than a column located adjacent to a water way (Tide = 1), however both of those catagories deteriorate much quicker than for bridges not located near a waterway (Tide = 0). Contrary to intuition, concrete columns located in a waterway were shown to deteriorate slower than both of the other classifications (F=1.36). Further investigation into this irregularity should be conducted. The frequency and urgency of repairs to columns located in these environments could be one possible explanation.

Brackish Waters

Table 27. Key Elements - Brackish Waters

Element		Brackish Waters							
Number	Name	Tide = 0				Tide = 1	Tide = 2	Tide = 3	Tide = 4
		T11	Beta	T22	T33	F	F	F	F
204	Prestressed Concrete Column	102.95	1.00	16.94	2.31	0.77	1.00	1.00	0.65
205	Concrete Column	59.87	2.27	6.40	500.00	0.94	1.00	1.28	2.00
215	Concrete Abutment	150.61	1.00	46.63	309.71	0.95	1.09	0.72	1.01

Tide = 0, 1, 2, 3 and 4 were created with the expectation of Tide = 0 being a benign environment and Tide = 4 being the most severe environment. Mixed results were found for this environemnt, particularly for bridges classified as Tide = 2 and Tide = 3 whose modification factors do not become worse in what is assumed to be a harsh environment. As seen in Figure 22, Tides 2, 3, and 4 each have a small number of unique bridges. The presence of brackish waters may not be the only influential environment on these small subsets of bridges.

It was also noted that an identical subset of bridges are classified as Tide = 0 in both the Waterways study and the Brackish Waters studies, this should result in baseline models for both environments having identical transision times. However, this was not achieved by the Markov-

Weibull optimization function. Upon closer inspection, the error terms (Eqn. 4) for both models were simlar and differences in transition times are determined to be negligable.

Freeze-Thaw

Table 28. Key Elements - Freeze-Thaw

Element		Freeze-Thaw Cycles				
Number	Name	Low				High
		T11	Beta	T22	T33	F
12	Concrete Deck	28.35	1.24	79.29	122.31	1.38
205	Concrete Column	57.30	2.41	6.53	500.00	1.23
215	Concrete Abutment	148.77	1.00	53.13	353.88	1.00
234	Concrete Pier Cap	67.46	2.10	7.60	500.00	1.06

All key elements for the Freeze-Thaw environment suggest that a higher number of freeze-thaw cycles result in a slower deterioration of the element. This result is opposite of what was expected but could imply that bridges with a low number freeze-thaw cycles are more directly impacted by an alternative environment. Additionally, freeze-thaw cycle counts were fairly consistant throughout Virginia, thus bridges classified as having a high number of cycles were only marginally different than those with a low number of cycles.

Temperature Extremes

Table 29. Key Elements - Temperature Extremes

Element		Temperature Range				
Number	Name	Low				High
		T11	Beta	T22	T33	F
12	Concrete Deck	29.84	1.46	73.50	92.84	0.92
205	Concrete Column	59.36	2.37	7.56	500.00	0.93
215	Concrete Abutment	147.66	1.00	63.45	188.33	1.06
234	Concrete Pier Cap	69.31	2.07	8.92	500.00	0.93

Small changes to transition times were found using temperature range as a defining environment. However, most of the key elements behaved as anticipated and deteriorated quicker when exposed to a wider range of temperatures through out a year. A further investigation of this environment using a smaller subset of bridges in the most severe environemnt may result in more significant modification factors.

Coastal Areas

Table 30. Key Elements - Coastal Areas

Element		Coastal Areas				
		Not Coastal				Coastal
Number	Name	T11	Beta	T22	T33	F
12	Concrete Deck	31.54	1.08	68.68	140.64	0.71
107	Steel Girder	54.34	1.09	35.15	65.04	1.40
205	Concrete Column	54.55	2.40	7.83	500.00	1.16
215	Concrete Abutment	154.69	1.00	59.50	306.46	0.83
234	Concrete Pier Cap	66.98	1.97	7.86	500.00	1.12

Concrete decks (F=0.71) and abutments (F=0.83) located within 10 miles of the coastline were shown to deteriorate much faster than elements further away from the coast. Concrete columns (F=1.16), pier caps (F=1.12), and steel girders (F=1.40) were shown to deteriorate slower when located near the coastline. When compared with the results found in the District Practices study it was found that districts bordering the coast line (District 5, 6, and 9) exhibited similar modification factors indicating that two environments are likely coupled to one another.

Combined Effects

Iteratively reducing the sample size by adding environment restrictions is shown to provide a more accurate reflection of element deterioration than a state-wide model alone. To demonstrate these results, concrete abutments (element 215) were studied by continually adding additional environments until only a single unique bridge was left in the subset. In Figure 32 and Figure 33, graph (A-#) shows the observed average condition state, (B-#) shows the modeled average condition state, and (C-#) shows the quantity of records available for the subset of data, where # corresponds to the combined environment detailed in Table 31. Effective modification factors for combined environments were calculated using Eqn. 8.

Table 31. Combined Environment Statistics – Element 215

Study Number (#)	Environment	F	Effective F	Unique Bridges in Subset
1	District = 4	0.52	0.52	170
2	Study 1 + No Joint Present	0.92	0.48	33
3	Study 2 + Functional Class = Collector	1.27	0.61	15
4	Study 3 + Waterways = 1	0.95	0.58	12
5	Study 4 + Lateral Splash Zone	1.14	0.66	1

The average condition state is calculated as a weighted average of the element percentage in each condition state using Eqn. 10.

$$CS_{avg}^{x} = \frac{(1*\%CS1)+(2*\%CS2)+(3*\%CS3)+(4*\%CS4)}{100} \text{, where} \qquad \text{[Eqn. 10]}$$

CS_{avg}^{x} is the average condition state at age x

%CSn is the percentage of data in condition state n

For example, if a 20 year old element is observed to be 50% in CS1, 20% in CS2, 20% in CS3, and 10% in CS4, the element would have an average condition state rating of 1.90. This metric is not intended to be used in a bridge management setting but rather to help visualize the impacts of environment modification factors.

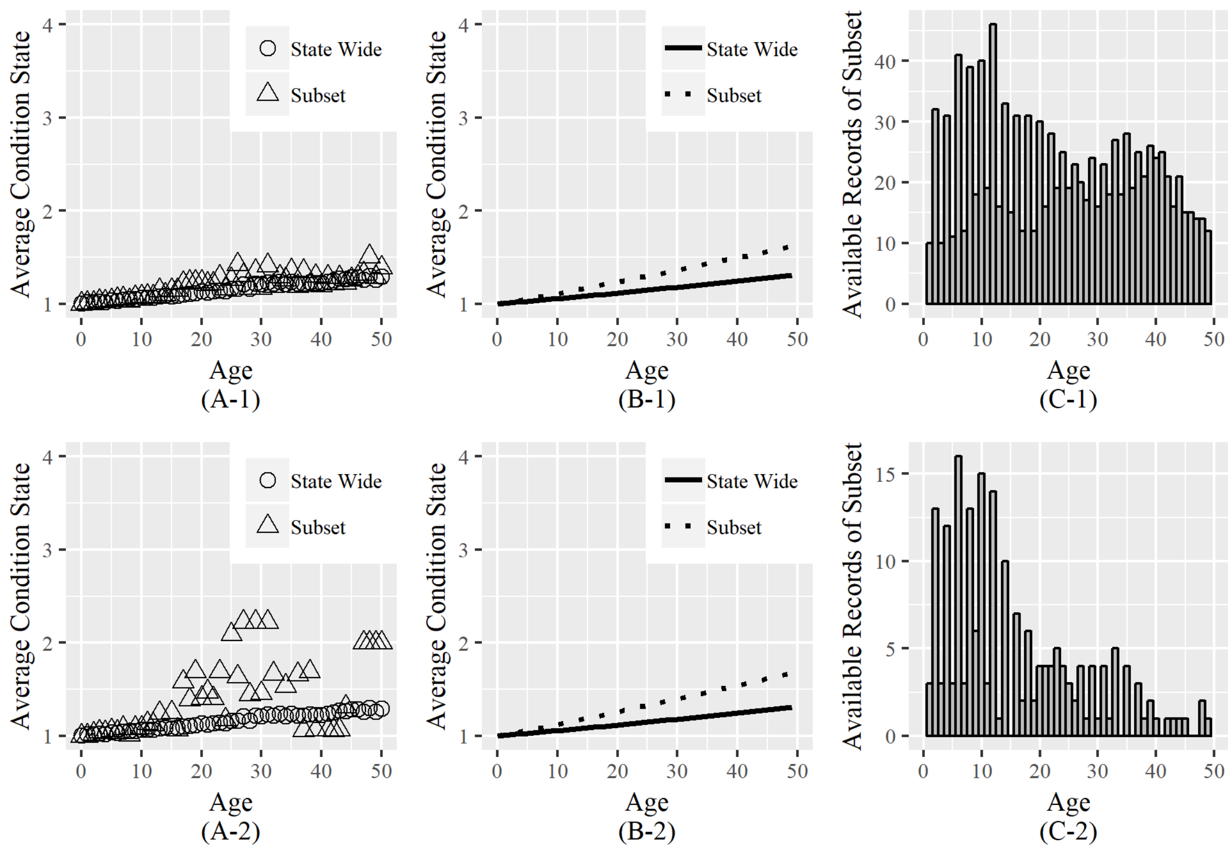

Figure 32. Combined Environment Effects

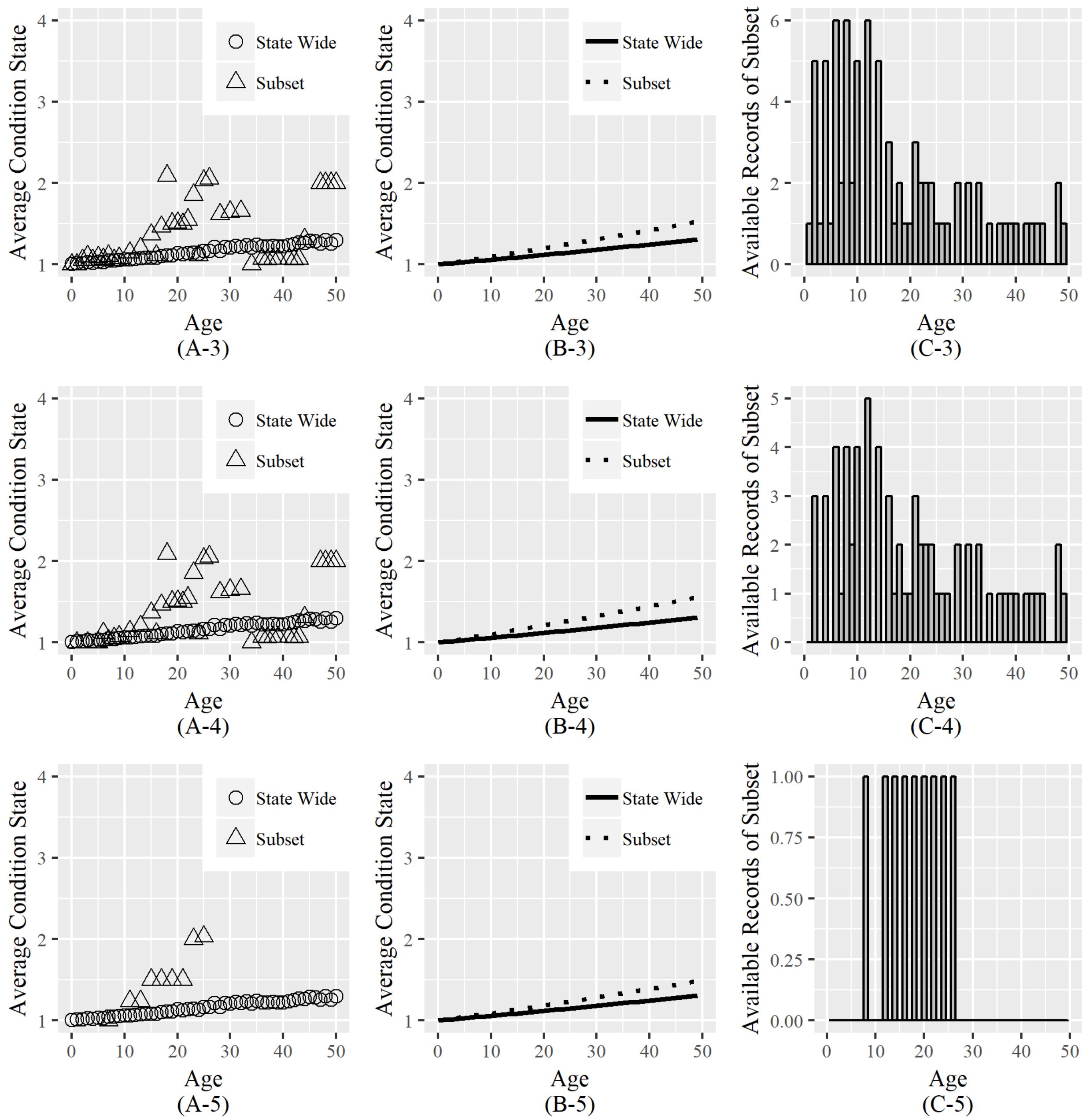

Figure 33. Combined Environment Effects (continued)

CONCLUSIONS

Model Fitting

- Many of the elements modeled were shown to have transition time 3 (T33) equal to 500 years. A length of time that is much longer than expected and may indicate an issue within the modeling approach or within the database itself. It was found that when bridges are decommissioned or demolished VDOT subsequently removes the bridges inspection records from the database. As a result of this, sets of critically useful data that describe a bridges gradual degradation from initial construction to the point of decommissioning is lost. This removal of bridges from the database results in an artificially low volume of data in Condition State 4 and subsequently inflates the modeled transition time T33.

Effectiveness of Identified Environments

Validated Environments

- Determining modification factors based on a bridges VDOT district revealed that select elements in district 1 and 6 deteriorate faster than the state-wide model whereas district 3 elements tend to deteriorate slower than the rest of the state.
- Traffic related environments such as functional classification and truck traffic volumes provided good results that were consistent with the initial assumptions of the technical advisory panel.
- Modification factors based on a bridges proximity to the coast line and subjectivity to being splashed by traffic passing beneath the bridge were successfully determined.
- The presence of a joints on a bridge was anticipated to drastically increase deterioration rates on select bridge elements. The models created in this study agreed that joints increase deterioration rates, however the impact of this environment was found to be less severe than initially anticipated. Limitations inherent to the inspection database are likely clouding the true scale of this environments impact.

Moderate Environments

- The waterways study was able quantify the harsh effects of having a bridge element located in water with reasonable success for select bridge elements.
- Freeze-thaw cycles and temperature ranges were both found to have relatively minor impacts on the deterioration of key bridge elements. Further research into these topics may reveal a more substantial pattern, however the data used to investigate these environments suggests that Virginia's weather patterns are relatively uniform across the entire state, therefore use of these environments are not expected to be fruitful for the purpose of bridge deterioration modeling.

Questionable Environments

- Limited success was had in the brackish waters study by way of combining salinity maps of the Chesapeake Bay with the waterways study. The study was expected to show that bridges directly exposed to high levels of salts deteriorate quicker than those in less concentrated water sources. The limited number of structures that qualify for the specific classifications did not always exhibit the expected behavior. Bridge groupings classified in the brackish water study may be closely coupled with other with other environments causing the unexpected behavior.

RECOMMENDATIONS

Future Work

- To better gauge the impact of a joint on subordinate bridge elements the joint presence study should be investigated further. The methodology adopted in the joint presence study relies on a predicted length of bridge joints based on the geometry of multiple bridge elements. This practice resulted in the exclusion of a large number of bridges from the database and may have other unintended consequences leading to the observed results. Simplification of the study by first investigating two-span bridges may provide cleaner results for the deterioration of substructure elements such as pier caps. Results from subsequent studies are likely to show more substantial modification factors for elements subjected to joints.
- Datasets used as baseline models were uniquely selected for each environment. Future work should investigate using a uniform dataset consisting of all state-wide data for each elements baseline model.
- Future studies should incorporate a method to measure model accuracy. This could be done by subdividing available inspection records into training and testing datasets.
- Further investigations to determine an optimum method of combining multiple modification factors should be completed. This report assumes all bridge environments have equal importance, however a weighted combination method may provide more a more accurate results.

Data Collection

- It is recommended that historical inspection records of decommissioned bridges be preserved for the purpose of future deterioration modeling. Retention of this data is expected to provide better estimates of the deterioration from Condition State 3 (poor health) to Condition State 4 (severe health).
- Local environments (such as joint presence, splash zones, or waterways) are only applicable to a portion of elements on any given bridge. With the current documentation, an element in a locally harsh environment is not distinguishable from an element absent from the environment. This merging of elements lessens the measurable impact of local environments. Ideally, future bridge inspection reports would be modified to identify which bridge elements are subjected to local environments.

Software Advancement

- Deterioration modeling within AASHTOWare Bridge Management restricts the modification of element deterioration to a single modification factor, F, that is applied uniformly to all transition times (T11, T22, and T33). The addition of transition specific modification factors in a future version of the widely used software would create a path to more accurately model transition times for bridges exposed to a defined environment.

BENEFITS AND IMPLEMENTATION PROSPECTS

Modification factors for multiple bridge elements were generated for each unique environment studied. The elements used in each study were selected individually based on the anticipated effect the environment would have on various bridge materials and bridge element types. Full tabular data for each environment is provided in Appendix B. One implementation strategy is to create a comprehensive list of all bridges in the database that can be cross-referenced with combinations of environments and their associated factors for each element. An individual modification factor for each bridges elements compatible with BrM can then be determined. Increasing the predictive capabilities of BrM will allow for more effective allocation of funds and improve VDOT's ability to create long term maintenance plans.

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APPENDIX A – NBI ELEMENT NUMBERS

Table A1. NBI Element Numbers (FHWA, 2014)

Element	Units	Element Number					
		Steel	Prestressed Concrete	Reinforced Concrete	Timber	Masonry	Other
Deck/Slab							
Deck/Slab	SF		13	12	31		60
Open Grid Deck	SF	28					
Concrete Filled Grid Deck	SF	29					
Corrugated or Orthotropic Deck	SF	30					
Slab	SF		39	38	54		65
Top Flange	SF		15	16			
Superstructure							
Closed Web/Box Girder	LF	102	104	105			106
Girder/Beam	LF	107	109	110	111		112
Stringer	LF	113	115	116	117		118
Truss	LF	120			135		136
Arch	LF	141	143	144	146	145	142
Main Cable	LF	147					
Secondary Cable	EA	148					149
Floor Beam	LF	152	154	155	156		157
Pin, Pin and Hanger Assembly	EA	161					
Gusset Plate	EA	162					
Substructure							
Column	EA	202	204	205	206		203
Column Tower (Trestle)	LF	207			208		
Pier Wall	LF			210	212	213	211
Abutment	LF	219		215	216	217	218
Pile Cap/Footing	LF			220			
Pile Cap/Footing	EA	225	226	227	228		229
Pier Cap	LF	231	233	234	235		236
Culvert							
Culvert	LF	240	245	241	242	244	243
Bridge Rail							
Bridge Rail	LF	330		331	332	334	333
Joint							
Strip Seal	LF	300					
Pourable	LF	301					
Compression	LF	302					
Assembly with Seal (Modular)	LF	303					

Open Grid Deck	LF	304
Assembly without Seal	LF	305
Other	LF	306
Bearing		
Elastomeric	EA	310
Movable (roller, sliding, etc.)	EA	311
Enclosed/Concealed	EA	312
Fixed	EA	313
Pot	EA	314
Disk	EA	315
Other	EA	316
Wearing Surfaces and Protective Coatings		
Wearing Surfaces	SF	510
Steel Protective Coating	SF	515
Concrete Protective Coating	SF	521

APPENDIX B – EFFECTS ENVIRONMENTAL FACTORS

Note: Bridge environments with an insufficient volume of data available are reported as N/A throughout Appendix B.

Table B1. Effects of District

	All Districts				District 1	District 2	District 3	District 4	District 5
Element	**T11**	**Beta**	**T22**	**T33**	**F**	**F**	**F**	**F**	**F**
12	22.17	1.06	71.26	22.46	0.97	1.53	1.28	1.94	0.79
13	26.89	1.67	30.63	267.82	0.69	0.79	1.44	2.70	1.39
28	18.53	9.40	0.10	500.00	1.00	1.00	1.00	1.00	0.82
29	24.84	2.09	2.64	500.00	1.00	1.00	1.00	1.00	1.00
30	31.36	1.00	2.38	7.46	0.05	0.92	1.07	10.54	N/A
31	16.62	1.00	54.66	21.81	0.72	3.38	N/A	6.12	0.73
38	31.79	2.01	38.72	33.12	0.89	1.14	1.14	1.33	1.42
39	44.70	2.23	51.45	198.73	1.30	N/A	1.38	0.92	1.00
54	500.00	9.54	137.67	294.80	1.00	1.00	1.00	1.00	1.00
102	61.25	1.01	23.74	18.64	2.53	N/A	1.00	1.31	0.62
104	177.17	1.05	121.90	14.84	1.64	0.42	1.40	2.36	0.67
105	224.80	1.00	156.00	32.20	1.40	N/A	N/A	N/A	1.12
107	44.64	1.23	25.12	44.60	0.75	0.81	1.79	0.76	1.14
109	269.96	1.24	30.43	423.26	1.38	2.33	1.71	2.31	0.69
110	221.54	1.33	34.08	92.56	0.79	0.87	1.98	1.60	0.93
111	52.68	2.47	500.00	176.18	N/A	1.00	1.00	1.00	1.00
113	41.00	1.00	28.24	9.51	0.46	1.04	6.04	34.50	0.85
202	13.70	1.00	19.78	13.75	0.33	0.99	N/A	N/A	5.14
204	36.64	1.95	24.50	500.00	0.93	1.00	1.00	1.35	1.31
205	56.46	2.50	6.62	500.00	0.80	0.86	1.31	1.03	1.49
206	51.87	1.87	32.15	35.28	1.00	1.00	0.83	0.56	1.22
210	85.55	1.80	25.20	500.00	0.73	1.23	2.04	0.56	1.03
215	126.25	1.00	49.69	174.97	0.86	1.41	1.28	0.52	0.81
216	65.60	1.33	12.93	42.12	1.41	1.00	1.12	0.43	1.96
217	63.82	1.00	65.09	64.30	1.15	1.00	1.50	0.84	1.00
231	29.82	1.68	6.59	9.15	0.15	0.68	2.81	2.62	1.26
234	67.64	2.02	7.95	500.00	0.87	0.79	1.49	0.95	1.25
235	116.00	1.54	32.85	25.42	N/A	2.65	0.97	0.35	1.84
300	53.63	1.00	44.65	42.84	1.96	2.09	5.93	0.52	2.55
301	44.61	1.00	13.51	63.71	0.52	0.68	2.12	0.57	2.68
302	40.33	1.00	35.75	80.33	1.15	0.88	1.14	1.01	0.90
303	57.25	1.00	97.81	89.81	3.01	2.06	2.07	0.44	0.73
304	54.35	1.00	111.67	47.24	0.60	0.62	3.28	0.47	N/A
310	44.53	1.27	283.20	71.13	1.57	1.28	1.32	1.53	1.02

311	28.62	1.68	156.31	60.06	1.06	1.04	1.51	1.04	0.90
312	64.19	6.28	38.04	365.99	1.00	1.00	1.03	2.61	1.00
313	30.87	1.46	213.57	33.30	1.06	1.13	1.98	1.07	0.74
314	145.69	1.00	38.78	16.87	0.25	8.03	N/A	2.01	0.76
315	61.80	1.16	499.97	201.61	1.00	1.00	1.00	1.00	0.62
330	500.00	1.00	26.17	5.63	0.96	0.62	0.77	1.44	1.63
331	114.00	1.15	140.35	449.42	0.73	0.91	1.79	0.50	1.11
332	123.80	1.00	29.00	38.38	0.81	1.50	5.42	0.50	3.37
333	60.65	4.45	46.53	275.10	0.95	0.91	0.99	1.13	1.10
334	149.12	1.00	31.48	49.89	0.65	0.75	1.06	0.47	2.13

Table B2. Effects of District (Continued)

	All Districts				District 6	District 7	District 8	District 9
Element	**T11**	**Beta**	**T22**	**T33**	**F**	**F**	**F**	**F**
12	22.17	1.06	71.26	22.46	0.96	1.66	1.25	0.77
13	26.89	1.67	30.63	267.82	0.85	1.00	1.34	1.53
28	18.53	9.40	0.10	500.00	1.00	1.00	1.00	2.25
29	24.84	2.09	2.64	500.00	N/A	1.00	1.00	1.00
30	31.36	1.00	2.38	7.46	1.00	N/A	1.00	N/A
31	16.62	1.00	54.66	21.81	1.88	3.45	0.32	3.04
38	31.79	2.01	38.72	33.12	0.99	0.98	1.10	1.39
39	44.70	2.23	51.45	198.73	0.75	3.76	1.23	0.76
54	500.00	9.54	137.67	294.80	1.00	1.00	1.00	1.00
102	61.25	1.01	23.74	18.64	N/A	1.00	1.00	1.00
104	177.17	1.05	121.90	14.84	0.94	0.43	3.78	1.49
105	224.80	1.00	156.00	32.20	1.00	4.07	1.71	0.49
107	44.64	1.23	25.12	44.60	0.85	0.78	0.55	1.63
109	269.96	1.24	30.43	423.26	0.97	1.31	3.90	2.79
110	221.54	1.33	34.08	92.56	2.72	0.45	2.89	2.52
111	52.68	2.47	500.00	176.18	1.00	1.00	1.00	1.00
113	41.00	1.00	28.24	9.51	N/A	1.00	0.70	4.66
202	13.70	1.00	19.78	13.75	N/A	1.06	1.00	0.58
204	36.64	1.95	24.50	500.00	1.00	1.00	1.00	1.23
205	56.46	2.50	6.62	500.00	0.83	0.94	1.02	0.88
206	51.87	1.87	32.15	35.28	1.00	2.20	1.00	N/A
210	85.55	1.80	25.20	500.00	0.92	1.21	1.06	0.74
215	126.25	1.00	49.69	174.97	0.91	0.80	1.48	0.97
216	65.60	1.33	12.93	42.12	0.95	0.37	1.00	1.00
217	63.82	1.00	65.09	64.30	0.58	0.62	0.64	1.62
231	29.82	1.68	6.59	9.15	0.66	0.53	1.00	0.68

234	67.64	2.02	7.95	500.00	0.96	1.05	0.99	0.86
235	116.00	1.54	32.85	25.42	0.97	0.60	1.00	N/A
300	53.63	1.00	44.65	42.84	1.04	1.40	0.51	0.44
301	44.61	1.00	13.51	63.71	0.51	0.68	0.98	0.57
302	40.33	1.00	35.75	80.33	0.54	1.45	1.22	0.93
303	57.25	1.00	97.81	89.81	0.91	0.79	2.66	1.06
304	54.35	1.00	111.67	47.24	11.16	3.16	6.41	0.91
310	44.53	1.27	283.20	71.13	0.97	1.13	1.46	1.25
311	28.62	1.68	156.31	60.06	0.83	0.89	1.17	0.99
312	64.19	6.28	38.04	365.99	1.00	1.00	2.00	0.85
313	30.87	1.46	213.57	33.30	1.02	0.93	1.28	1.03
314	145.69	1.00	38.78	16.87	0.26	1.00	1.26	3.10
315	61.80	1.16	499.97	201.61	N/A	1.00	1.00	1.17
330	500.00	1.00	26.17	5.63	12.04	5.29	7.70	0.68
331	114.00	1.15	140.35	449.42	0.48	0.92	2.29	2.05
332	123.80	1.00	29.00	38.38	1.45	0.97	1.05	1.33
333	60.65	4.45	46.53	275.10	0.71	1.18	1.31	0.95
334	149.12	1.00	31.48	49.89	1.00	1.64	1.72	0.69

Table B3. Effects of Functional Class

	1, 2, 6, 11, 12, 14				7, 8, 16, 17	9, 19
Element	**T11**	**Beta**	**T22**	**T33**	**F**	**F**
12	19.77	1.27	113.74	43.85	1.27	1.95
13	30.95	3.14	15.61	500.00	1.13	0.92
38	19.06	2.23	500.00	7.59	1.90	1.88
39	43.52	2.11	144.84	141.23	1.27	1.26
104	119.30	1.00	234.09	36.90	3.59	1.93
105	122.68	1.00	75.75	21.73	4.04	8.66
107	63.60	1.08	34.46	60.83	0.77	0.64
109	146.41	1.79	32.79	44.36	2.13	1.87
110	317.93	1.18	30.55	324.64	0.89	0.94
113	58.37	1.00	38.66	13.12	0.94	1.53
202	25.22	10.00	43.26	8.15	1.18	0.39
204	31.70	2.34	24.92	500.00	1.85	3.63
205	54.96	2.52	5.76	500.00	1.09	1.21
206	54.96	2.52	5.76	500.00	1.09	1.21
210	73.44	2.26	9.39	500.00	1.02	1.07
215	116.34	1.00	35.86	266.31	1.27	1.13
217	58.41	1.83	48.29	500.00	0.82	0.89
231	49.33	1.13	8.72	28.66	0.73	0.48
234	60.72	2.25	5.82	500.00	1.16	1.50
310	38.42	1.30	222.51	19.61	1.86	2.21
311	27.05	1.88	149.33	37.64	1.19	1.41
312	49.96	3.13	255.90	6.60	2.19	2.51
313	27.80	1.58	201.17	20.38	1.40	1.61
314	198.41	1.00	50.94	11.10	0.19	0.58
330	500.00	1.00	39.80	8.87	2.24	0.70
331	96.07	1.29	135.48	240.88	1.11	1.17
332	30.53	2.37	58.47	25.69	2.64	2.81
333	53.08	7.75	31.96	440.68	1.10	1.05
334	338.29	1.00	40.36	16.59	0.49	0.29

Table B4. Effects of Truck Traffic

	Low ADTT				High ADTT
Element	**T11**	**Beta**	**T22**	**T33**	**F**
12	28.77	1.08	90.62	143.76	0.85
107	62.42	1.15	34.07	82.76	0.64
109	500.00	1.05	32.61	43.22	3.00
110	211.20	1.43	47.16	329.73	1.11
205	61.75	2.23	8.15	500.00	0.77
210	82.55	1.89	22.28	500.00	1.42
215	147.25	1.00	56.58	266.21	1.07
234	74.65	1.92	9.46	500.00	0.67
310	52.99	1.19	260.36	29.46	0.95
311	30.67	1.65	160.58	64.03	0.73
313	34.11	1.51	217.95	37.61	0.91
330	500.00	1.03	42.11	14.58	2.25
331	143.56	1.03	141.18	500.00	0.90
333	59.87	4.85	51.37	327.81	0.96
334	141.35	1.00	34.41	124.25	19.80

Table B5. Effects of Joint Presence

	Joint				No Joint
Element	**T11**	**Beta**	**T22**	**T33**	**F**
102	74.30	1.01	50.65	28.83	0.60
104	169.07	1.03	163.25	27.04	4.48
105	148.59	1.62	170.89	362.87	0.39
107	64.54	1.13	35.90	87.02	1.10
109	500.00	1.13	23.82	39.00	1.11
110	166.15	1.78	26.30	213.54	0.63
215	152.11	1.00	45.97	298.81	0.92
217	45.23	1.79	26.93	150.19	1.48
231	41.47	1.72	13.33	43.77	2.12
233	227.90	1.00	110.24	19.03	1.95
234	67.39	2.04	8.04	500.00	1.11
310	45.33	1.36	280.29	27.38	1.65
311	29.45	1.80	157.61	51.01	1.23
312	53.90	10.00	11.32	500.00	1.00
313	32.73	1.56	218.84	31.06	1.33
314	227.61	1.00	56.84	9.26	0.23

Table B6. Effects of Lateral Splash Zone

	High Clearance				Low Clearance
Element	**T11**	**Beta**	**T22**	**T33**	**F**
204	35.22	7.39	500.00	8.86	0.47
205	56.47	2.53	7.09	500.00	0.95
210	60.69	2.46	8.38	500.00	1.02
215	108.12	1.00	41.84	87.01	1.14
217	455.64	9.72	327.59	249.07	1.00
231	242.60	1.00	73.81	6.75	0.29
234	74.73	1.53	9.12	500.00	0.91

Table B7. Effects of Vertical Splash Zone

	High Clearance				Low Clearance
Element	**T11**	**Beta**	**T22**	**T33**	**F**
12	23.84	1.32	54.76	161.30	1.25
13	33.88	8.32	11.66	356.38	1.02
38	29.13	3.51	57.17	0.10	0.75
105	84.01	1.00	93.71	12.96	4.45
107	58.04	1.00	30.36	53.83	1.47
109	500.00	1.36	20.33	123.75	0.48
110	379.20	1.00	30.74	68.78	0.96
113	58.91	1.90	15.69	39.25	0.88
205	56.10	2.48	6.03	500.00	1.02
210	88.64	1.69	9.19	500.00	0.76
215	106.80	1.00	33.29	128.75	0.83
234	66.52	1.81	7.43	500.00	0.82

Table B8. Effects of Waterways

	Tide = 0				Tide =1	Tide =2
Element	**T11**	**Beta**	**T22**	**T33**	**F**	**F**
12	28.21	1.00	82.97	129.66	1.07	0.96
13	31.23	1.97	21.42	196.02	0.99	0.78
31	22.06	1.00	47.24	25.05	1.32	1.78
107	59.48	1.08	34.87	81.31	1.11	1.51
109	338.72	1.64	11.29	308.44	0.84	0.38
110	358.97	1.07	34.53	314.83	1.18	1.19
113	115.87	1.00	60.20	37.67	0.89	0.19
204	105.73	1.00	24.39	156.59	0.57	0.74
205	59.87	2.27	6.40	500.00	0.94	1.36
206	52.75	2.47	23.10	273.27	1.14	0.87
210	103.66	1.56	17.91	500.00	0.87	1.81
215	150.61	1.00	46.59	307.48	0.95	0.81
216	72.83	1.68	13.44	145.72	0.82	1.00
234	67.27	2.00	7.09	500.00	1.08	1.30
235	223.92	1.03	40.54	80.14	1.00	0.53
310	49.59	2.54	244.81	74.64	0.93	0.82
311	32.20	1.59	234.28	102.74	0.99	0.65
313	37.99	1.62	412.86	63.14	0.95	0.54

Table B9. Effects of Brackish Waters

	Tide = 0				Tide = 1	Tide = 2	Tide = 3	Tide = 4
Element	**T11**	**Beta**	**T22**	**T33**	**F**	**F**	**F**	**F**
12	28.21	1.00	82.99	129.68	1.07	0.95	0.95	0.01
13	31.22	1.98	21.29	176.99	0.99	1.00	0.80	1.05
31	22.06	1.00	47.24	25.05	1.32	0.22	1.82	1.00
107	59.46	1.08	34.88	81.31	1.11	1.75	0.86	1.75
109	342.33	1.63	11.34	326.55	0.84	1.00	0.40	0.50
110	348.08	1.08	33.41	179.79	1.18	1.00	2.09	0.59
113	115.87	1.00	60.21	37.67	0.89	1.00	0.19	1.00
204	102.95	1.00	16.94	2.31	0.77	1.00	1.00	0.65
205	59.87	2.27	6.40	500.00	0.94	1.00	1.28	2.00
206	52.71	2.47	22.90	123.98	1.14	0.59	0.93	1.00
210	103.71	1.56	17.91	500.00	0.87	0.76	1.44	2.92
215	150.61	1.00	46.63	309.71	0.95	1.09	0.72	1.01
216	72.82	1.68	13.43	144.98	0.82	0.56	1.20	1.20
234	67.24	2.00	7.09	500.00	1.08	0.97	1.24	2.12
235	228.04	1.02	41.02	85.47	0.99	0.18	0.92	2.00
310	49.59	2.54	244.88	75.39	0.93	1.00	0.86	0.94
311	32.21	1.59	236.11	119.06	0.99	0.38	0.63	0.90
313	37.99	1.62	389.22	51.80	0.95	0.74	0.52	0.69

Table B10. Effects of Freeze-Thaw Cycles

	Low Freeze-Thaw				High Freeze-Thaw
Element	**T11**	**Beta**	**T22**	**T33**	**F**
12	28.35	1.24	79.29	122.31	1.38
13	27.73	1.53	35.21	114.10	1.00
31	23.42	1.00	50.88	25.44	1.39
38	35.65	1.90	44.99	170.12	0.93
39	54.20	2.11	48.07	72.32	0.94
104	204.89	1.02	158.54	28.80	3.28
107	62.65	1.14	33.77	82.64	0.83
109	500.00	1.17	25.00	94.99	1.52
110	264.73	1.24	41.59	236.60	1.89
204	36.43	1.91	25.93	500.00	1.26
205	57.30	2.41	6.53	500.00	1.23
206	60.89	1.94	30.69	86.57	1.06
210	88.45	1.74	24.56	500.00	1.01
211	52.83	2.20	438.21	500.00	0.90
215	148.77	1.00	53.13	353.88	1.00
216	70.78	1.67	17.37	112.20	0.80
217	77.66	1.00	80.02	83.38	0.95
234	67.46	2.10	7.60	500.00	1.06
235	122.54	1.60	39.57	80.90	0.88
310	47.79	2.69	197.13	37.30	0.90
311	30.33	1.69	177.92	55.75	0.88
312	58.25	9.38	23.11	500.00	1.00
313	35.27	1.72	225.24	37.39	0.85
330	500.00	1.00	40.77	14.26	2.18
331	127.89	1.09	160.67	499.99	1.10
332	129.75	1.00	26.24	41.39	1.64
333	91.31	1.92	77.81	500.00	1.44
334	153.13	1.00	35.18	151.89	0.80

Table B11. Effects of Temperature Extremes

	Low Temp. Range				High Temp. Range
Element	**T11**	**Beta**	**T22**	**T33**	**F**
12	29.84	1.46	73.50	92.84	0.92
13	23.45	1.21	32.63	53.43	1.30
31	16.74	1.00	60.08	19.15	2.50
38	34.49	1.83	48.69	287.80	1.12
39	49.70	2.06	46.18	86.81	2.45
104	176.61	1.00	136.27	24.27	2.05
105	68.18	3.78	117.17	109.11	0.97
107	64.48	1.23	27.40	94.75	0.80
109	500.00	1.06	23.64	51.21	2.21
110	223.37	1.38	36.97	341.31	1.07
113	62.07	1.00	55.75	16.13	1.78
202	53.90	1.00	11.82	40.68	0.39
205	59.36	2.37	7.56	500.00	0.93
206	59.56	1.90	37.06	95.69	1.57
210	99.89	1.61	40.39	500.00	0.92
211	66.39	1.00	224.47	21.79	0.67
215	147.66	1.00	63.45	188.33	1.06
216	76.68	1.50	18.53	133.66	0.61
217	78.11	1.00	87.14	107.34	0.82
231	41.42	3.79	14.55	15.76	1.06
234	69.31	2.07	8.92	500.00	0.93
235	108.33	1.71	38.23	139.72	1.58
310	45.36	1.32	282.29	28.41	1.31
311	29.80	1.79	141.81	55.23	1.05
312	61.50	7.26	30.49	500.00	4.01
313	32.40	1.59	202.95	37.54	1.16
314	127.60	1.00	43.16	17.65	1.74
315	27.76	10.00	500.00	34.11	1.83
330	433.89	1.17	78.28	43.75	1.17
331	114.44	1.11	187.32	500.00	1.62
332	180.11	1.00	30.88	66.47	0.60
333	52.52	7.45	32.09	317.00	1.15
334	155.34	1.00	25.90	117.82	1.00

Table B12. Effects of Coastal Areas

	Not Coastal				Coastal
Element	**T11**	**Beta**	**T22**	**T33**	**F**
12	31.54	1.08	68.68	140.64	0.71
13	24.46	1.34	28.17	249.40	1.56
31	24.16	1.00	43.77	26.19	0.90
38	35.73	1.91	51.63	118.77	0.74
39	68.92	1.99	23.81	91.51	0.51
104	229.95	1.00	141.11	21.34	0.69
105	74.35	3.67	92.43	345.64	0.85
107	54.34	1.09	35.15	65.04	1.40
109	500.00	1.21	20.75	69.97	0.70
110	259.89	1.25	41.63	306.45	2.28
113	83.85	1.34	65.71	34.07	0.48
202	12.08	1.00	36.63	19.24	26.76
204	52.47	9.88	27.69	36.85	0.70
205	54.55	2.40	7.83	500.00	1.16
210	105.95	1.56	29.84	500.00	0.66
215	154.69	1.00	59.50	306.46	0.83
216	82.91	1.18	19.77	127.89	1.34
217	80.82	1.00	86.26	119.23	0.44
231	42.98	1.58	39.26	5.82	1.52
234	66.98	1.97	7.86	500.00	1.12
310	50.73	2.30	335.83	37.94	0.74
311	32.62	1.55	169.63	85.62	0.82
312	80.51	4.41	500.00	161.71	0.69
313	38.50	1.60	224.56	44.77	0.71
314	132.09	1.00	17.84	29.02	0.87
330	500.00	1.19	64.74	31.82	0.42
331	154.48	1.02	147.91	500.00	0.73
332	142.64	1.00	27.08	47.48	0.89
333	79.47	2.93	44.36	500.00	0.74
334	149.91	1.00	29.36	145.92	1.18