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16. Abstract			
Bridge components are affected by loads and environmental stressors, deteriorating faster or even collapse without effective maintenance and rehabilitation strategies. Furthermore, wet-dry cycling and higher concentrations of chlorides in coastal areas accelerates the deterioration process of bridges while increasing the frequency of maintenance and cost of the repairs. To address this problem, innovative materials like Ultra High-Performance Concrete (UHPC) should be considered in the development and implementation of maintenance and rehabilitation strategies. A comparison of conventional and UHPC applications for bridge repairs is presented in this research study. A step-by-step framework is proposed to select the repair material based on performance requirements, life-expectancy, and life-cycle cost. Durability, mechanical properties, and construction factors are considered to evaluate the concrete performance. The life expectancy models for conventional concrete and UHPC are based on chloride corrosion and Monte Carlo simulation. The Life Cycle Cost Analysis (LCCA) methodology is part of the framework and includes the quantification of agency and user costs over time. A case study is presented to demonstrate the applicability of the LCCA methodology, and it was found that the use of UHPC can result in a significant reduction in the total life cycle cost. Products from this research aims to support funding allocation decisions at both network and project management level. The step-by-step framework for selecting concrete material and LCCA methodology are tools developed in this study to identify cost-effective retrofitting techniques to preserve bridges in a "State of Good repair".			
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LIFE-CYCLE COST ANALYSIS OF ULTRA HIGH-PERFORMANCE CONCRETE (UHPC) IN RETROFITTING TECHNIQUES FOR ABC PROJECTS

Final Report

August 2022

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Sponsored by
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DISCLAIMER

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The collaboration of the Research Advisory Panel members in this research project is also acknowledged by the authors. The RAP provided feedback during the development of the research and project information for the LCCA case study presented in this final report.

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CHAPTER 1: INTRODUCTION

1.1 Project Motivation

Ultra-High-Performance Concrete (UHPC) is an innovative material with the potential to become a viable alternative for improving the sustainability of infrastructure components. UHPC is an exceptionally cementitious material, durable against freeze-thaw attack and permeation of gases and liquids. It has a low water-to-cement ratio and a low maximum grain size diameter with the addition of pozzolanic filler materials like silica fume.

Recognizing the potential benefits of UHPC applications, decision-making tools for determining when and how to use UHPC are needed. Life-cycle cost analysis (LCCA) is one of the tools that can assist to compare treatment solutions for bridge maintenance strategies to preserve Accelerated Bridge Construction (ABC) projects in good condition. To determine the best cost-effective maintenance strategy, it is important to understand the deterioration mechanism of bridge components. For reinforced concrete elements, the deterioration process can be modeled as a function of the steel corrosion affecting reinforced concrete bridge elements. LCCA can quantify the total costs of al investment alternatives using software tools with deterministic or probabilistic approaches.

The life cycle of ABC projects includes several phases: planning, design, construction, maintenance, rehabilitation, reconstruction, or recycling. Most of the research studies for ABC projects have been focused on the design and construction phases, although maintenance and rehabilitation activities are important to preserve bridges in good condition. This research is focused on the development and implementation of a life-cycle cost performance-based methodology to incorporate UHPC applications for retrofitting techniques in ABC projects.

It is expected that the use of innovative materials and timely maintenance strategies will contribute to extend the life expectancy of bridges using ABC systems. Products from this research aims to support decisions at the network and project management levels.

Problem Statement

Accelerated Bridge Construction (ABC) projects combine construction methods and innovative systems to reduce the time to build new bridges and to rehabilitate old bridge components. Planning, design, construction, maintenance, rehabilitation, reconstruction, and/or recycling activities will affect the performance of ABC projects in their service life. Research studies conducted for ABC projects have been mainly focused on design and construction, although maintenance and rehabilitation are critical to preserve a bridge in good performance.

Bridge components are affected by loads and environmental stressors deteriorating faster or even collapse without effective maintenance and rehabilitation strategies. Previous research efforts have identified deterioration mechanisms due to steel depassivation by chloride ions. Furthermore, wet-dry cycling and higher concentrations of chlorides in coastal areas accelerate the deterioration process reducing the life expectancy of bridges while increasing the frequency and cost of the repairs. Therefore, more innovative durable repair materials should be considered in the formulation of maintenance and rehabilitation strategies. UHPC compressive and tensile strength, ductility and modulus of elasticity are notably higher than normal strength concrete, increasing its durability under environmental stressors. UHPC has higher initial costs, but the total life-cycle costs should be lower when compared to conventional concrete. Although, this seems to be a reasonable hypothesis, the problem is to how to analyze and quantify the life-cycle costs of UHPC retrofitting applications in ABC projects.

1.2 Research Objectives and Tasks

Main Research Objectives

The main objective of this research is to quantify the potential benefits of UHPC applications in retrofitting techniques to adopt long-term cost-effective maintenance strategies for ABC projects. To accomplish this objective, a comprehensive research approach is followed in the development of the study. The approach begins investigating previous efforts seeking potential applications of innovative materials, retrofitting methods, and software tools to conduct life-cycle cost analysis (LCCA). Findings from this investigation are the foundation to develop a step-by-step LCCA methodology to support the decision-making process for ABC projects at the network and project management level.

The research objectives are very well aligned with research efforts conducted at the Accelerated Bridge Construction University Transportation Center (ABC-UTC). The proposed research study contributes with the following ABC-UTC objectives: (a) to foster the use of new materials and timely maintenance strategies to increase the life expectancy of bridges built using ABC systems, (b) to mitigate the effects of climate change on the deterioration of ABC components throughout their service life, (c) to develop decision-making analytical tools to support ABC project implementation by local agencies, (d) to promote ABC projects implementation through research that incorporates innovative repair materials to improve their performance over time.

Research Project Tasks

A comprehensive state-of-practice review followed by the development of life expectancy and cost effectiveness models for UHPC retrofitting treatments, development of a step-by-step LCCA methodology, and preparation of a case study was conducted in this research. Figure 1 shows the task dependencies and tasks relationships.

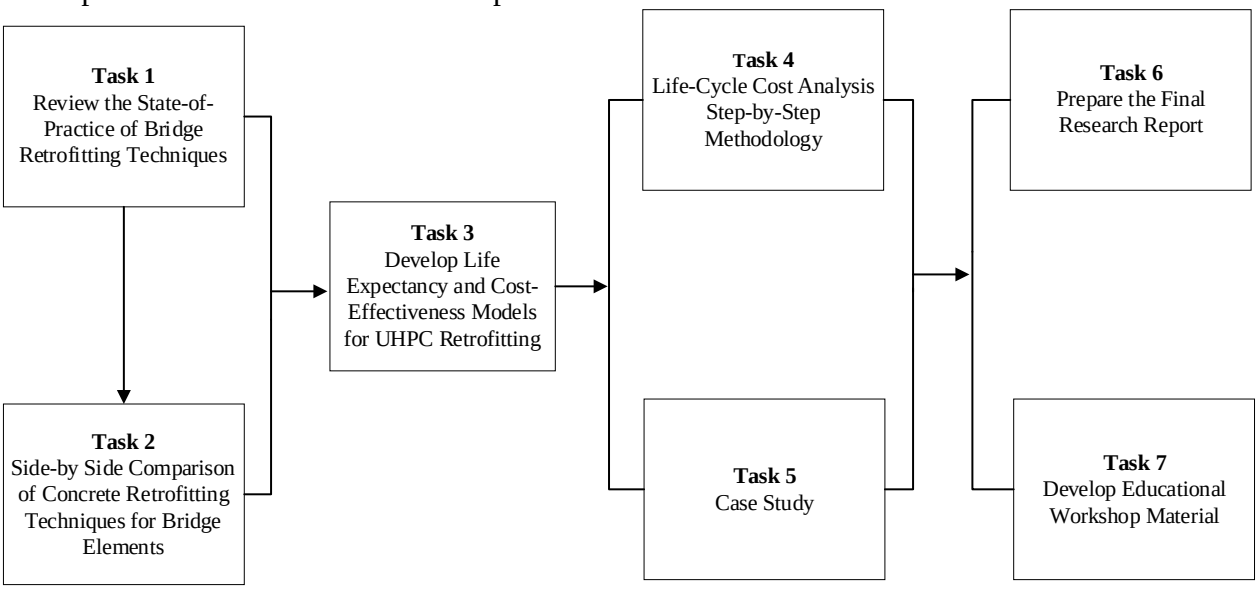

Figure 1. Overview of the research work plan.

In summary, the research work was composed of seven tasks:

Task 1: Review the State-of-Practice of Bridge Retrofitting Techniques
Task 2: Side-by-Side Comparison of Concrete Retrofitting Techniques for Bridge Elements.
Task 3: Develop Life Expectancy and Cost-Effectiveness Models for UHPC Retrofitting
Task 4: Life-Cycle Cost Analysis Step-by-Step Methodology
Task 5: Case Study

Task 6: Prepare the Final Research Report
Task 7: Develop Educational Workshop Material

All tasks were successfully completed as scheduled and the results are documented in this Final report.

1.3 Research Advisory Panel (RAP)

FDOT engineers contributed to this project providing relevant information with examples of UHPC applications in ABC projects. The following FDOT engineers collaborated in this effort:

Felix Padilla, P.E.
FDOT Office of Maintenance - State Structures Maintenance Engineer

Kellie Spurgeon, P.E.
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Timothy Barnard, P.E.
Engineering Section Manager at Florida Department of Transportation

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1.4 Report Overview

Table 1 provides an overview of the content of the Chapters included in this final report.

Table 1. Overview of the report.

Chapter	Chapter Overview
Chapter 1	Explains the project motivation, research objectives, and tasks; and presents an overview of the report.
Chapter 2	Provides a side-by-side comparison of the concrete retrofitting techniques including factors affecting life expectancy and treatment cost-effectiveness. This side-by-side comparison identifies relevant parameters for modeling UHPC performance for retrofitting techniques in ABC projects.
Chapter 3	Summarizes deterministic and probabilistic life expectancy models for concrete bridges. A hybrid chloride corrosion-based model combined with Monte Carlo simulation is presented in this Chapter to simulate the life expectancy of UHPC.
Chapter 4	Describes the Life Cycle Cost Analysis (LCCA) methodology for UHPC retrofitting applications for ABC projects. The LCCA methodology allows compare maintenance and rehabilitation strategies with conventional concrete and UHPC.
Chapter 5	Includes a comprehensive case study that demonstrates the LCCA methodology developed in this research work. For the case study, conventional in cast in place concrete deck slab was compared to precast deck slab with UHPC closure joints. Cost references were from previous research and recommendations from FDOT engineers.
Chapter 6	Includes the conclusions and recommendations from the research work.

1.5 Results and Specific Deliverables

This research project has developed a LCCA methodology for UHPC retrofitting applications for ABC projects. Results of the research work include the following:

- Life-expectancy performance models for UHPC retrofitting techniques.
- Comparison of life-cycle costs of UHPC alternatives to conventional techniques with examples for ABC projects.
- Recommendations to improve the cost-effectiveness of retrofitting techniques using UHPC in ABC maintenance and rehabilitation strategies.

Figure 2 shows a schematic diagram with the deliverables of the research. There are three main deliverables as the foundation of the LCCA methodology: (a) comparison of retrofitting techniques, (b) framework for selecting the appropriate concrete material, and (c) life expectancy models. The LCCA methodology is the main deliverable of this research illustrated with a case study. Finally, recommendations to improve the cost effectiveness of retrofitting techniques using UHPC are provided in the research deliverables.

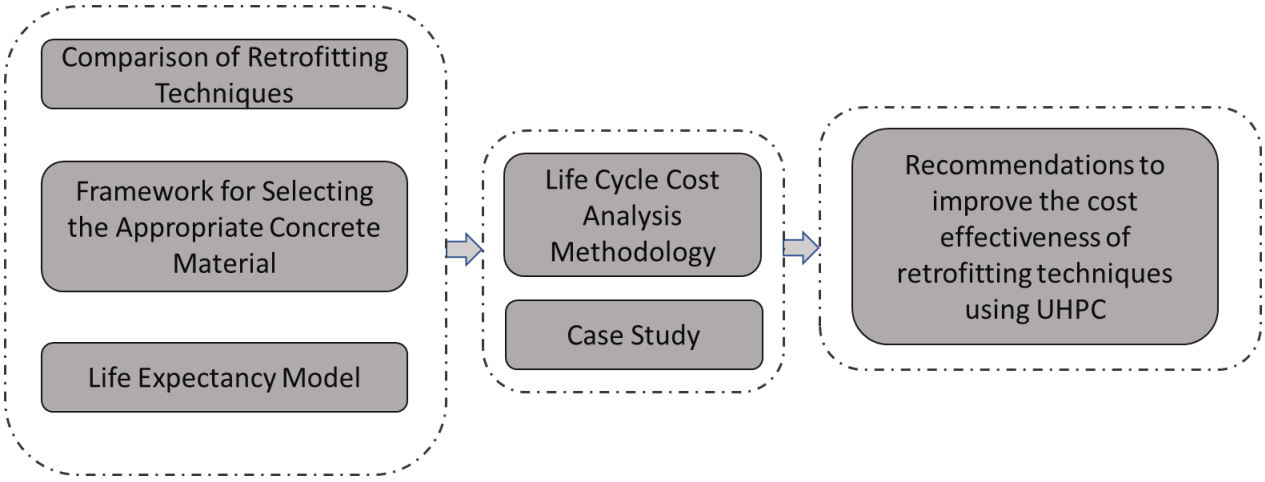


Figure 2. Schematic diagram of the main deliverables of the research.

A user guide complements the final report with a summary of the research findings and guidelines for implementation. Training educational material is also available with presentations about the content of this report.

CHAPTER 2: SIDE-BY- SIDE COMPARISON OF CONCRETE RETROFITTING TECHNIQUES FOR BRIDGE ELEMENTS

This Chapter describes concrete retrofitting techniques for bridge elements and provides recommendations for the selection of repair materials. The conventional concrete and UHPC applications for Accelerated Bridge Construction (ABC) are compared as alternative solutions for specific bridge elements (e.g., bridge deck, girder, column). Life-cycle Cost Analysis (LCCA) involves the comparison of alternatives and knowledge of the mechanical and durability properties of conventional concrete and UHPC is required to identify technical feasible solutions. The selection of the repair materials is critical in ABC projects. This Chapter provides guidelines based on performance criteria to develop retrofitting alternatives for ABC projects.

2.1 Comparison of Retrofitting Techniques

Environmental and traffic loads affect bridge performance in terms of functionality, structural integrity, and safety. There are many alternatives when choosing a retrofitting technique for a bridge component. The retrofitting strategy can be to repair or to replace the damaged bridge elements. The general retrofitting methods used to restore the capacity of bridges are: adding members, adding supports, reducing dead load, providing continuity, providing composite action, applying external post-tensioning, increasing member cross section, modifying load paths, adding lateral supports or stiffeners (Klaiber et al., 1988).

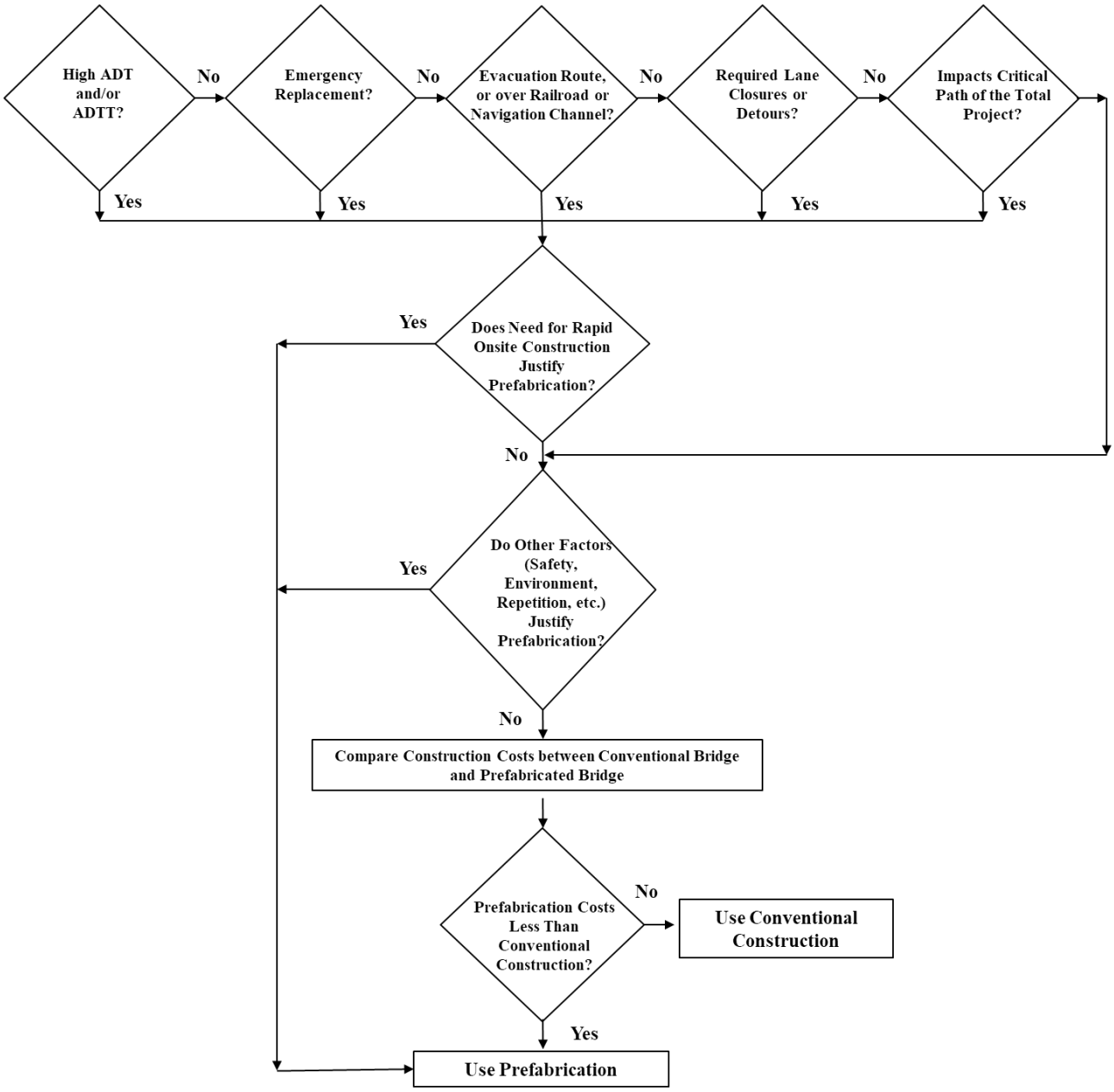
ABC offer alternative solutions to conventional construction. Repairs combined with replacement of bridge elements is common in ABC rehabilitation techniques. Table 2 shows ABC repair and replacement techniques that have been implemented across the United State.

Table 2. Summary of ABC techniques for bridge components.

Bridge Component	ABC Retrofitting Techniques	
	Repair	**Replacement**
Abutments	Anchors Diaphragm wall Reinforced concrete strut	Precast concrete abutments and Foundation Elements
Deck	Overlay Patch Repair	Exodermic deck panels Full-depth precast prestressed concrete deck panels Open steel grids Partial-depth precast prestressed concrete deck panels Precast Deck Panels
Deck Joint	Expansion Joints Deck Closure pours	UHPC Link Slab
Girder	Externally bonded FRP	Inverset Panels Precast modified beam in slab bridge system Precast concrete I-girders Precast girders with spliced details Precast bulb-tee girders Precast spread box girders Steel Girder
Pier and Column	Cement injections Drilled piles Fiberglass jackets Reinforced Concrete Jacket Sheet pile wall addition	Pre cast piers

Note: This table summarizes technical information from several sources (Aktan and Attanayake, 2013), (Wipf et al. 2004), (Khayat and Valipour 2018), (Rollins 2015).

Innovative precast bridge elements have already been used by several Departments of Transportation for the replacement of bridge elements. Precast concrete deck panels are the most popular used bridge elements in ABC projects. Figure 3 shows a flowchart with decision-making criteria to evaluate if prefabrication should be considered to accelerate the bridge construction process (Tang 2006).



Source: Tang 2006

Figure 3. Decision-making flowchart to use prefabricated elements in bridge construction.

2.2 Performance-Based Selection of Retrofitting Techniques

To select a retrofitting technique is important to understand the cause and mode of failure of the bridge component. The process should begin with a failure investigation and diagnosis about the type of failure, severity, and magnitude of the damage. The failure causes are related to the performance criteria which ultimately lead to the design parameters of retrofitting material. For example, leaking deck joints and spray from vehicles underneath can damage the substructure elements of the bridge. If the abutments are found to be deteriorated, a full bridge replacement may be needed because it is difficult to replace abutments without affecting the other elements.

A description for retrofitting techniques for the main bridge elements follows.

Column

Among conventional methods, concrete jacketing is the most common technique. The deterioration of concrete jacketing largely depends on the bond behavior between substrate and retrofitted concrete. UHPC can be used instead of conventional concrete for shell encapsulation of deteriorated bridge columns. ABC techniques offer alternatives for the rehabilitation of columns and pier caps. For example, in a situation where the existing bridge pier is support on a spread footing, the prefabricated new pier can be incorporated using Slide in Bridge Construction (SIBC) (Aktan and Attanayake 2017). Table 3 shows a comparison of conventional and accelerated construction techniques as alternatives to repair or replace bridge columns.

Table 3. Comparison of bridge column retrofitting techniques.

Factors Affecting Life Expectancy	Conventional Technology		Accelerated Construction	
	Concrete Jacketing	**Steel Jacketing**	**UHPC Shell**	**Pre-Cast Piers**
Construction Method	The section of the column is enlarged by casting a new reinforced concrete/mortar section over a part or the entire length of the column.	RC section is enlarged by welding or bolting it with a steel section	Shell encapsulation of column with UHPC	Conventional material strengths and design procedures are used.
Deterioration Factors	bonding failure to existing concrete	Bond slip of the dowel bars	Lack of proper curing in hot climate can lead to shrinkage crack very quickly	Damage during handling and transportation
Advantages	Lateral capacity is strengthened Familiarity of practicing engineers with the material Increases both strength and ductility	Resist shear force Ductile material	The seismic performance of bridge columns with deficient lap splices in the plastic hinge zone could be significantly improved by UHPC jacketing UHPC has low permeability, thus less susceptible to chloride diffusion	Great durability in corrosive environments. Round columns show better seismic performance compared to other sections.
Limitations	Labor intensive and time consuming due to formwork installation Change in cross-sectional size due to corrosion Increase in ductility is small due to brittle nature of concrete	Steel prone to rusting and corrosion Makes structure heavy Weight	High initial cost Proper curing is needed for UHPC	Shipping and handling may be a limitation depending on the height and weight of column.

Note: This table summarizes technical information from several sources (Aktan and Attanayake 2013), (FHWA 2019), (Branco and Silva, 2003), (Farzad et al. 2020).

Beam/Girder

Conventional techniques for strengthening girder/beam are external prestressing repair with CFRP/FRP sheets. The limitations with these techniques are the materials used in the retrofit. The tendons used in prestressing are prone to corrosion if they are exposed. The CFRP debonding from existing concrete is also common, but the CFRP material itself is durable. A variety of girder designs have been implemented for ABC to comply with bridge type, site constraints, and bridge span.

Table 4 shows a comparison of conventional and accelerated constructions techniques as alternatives to repair or replace beam/girders.

Table 4. Comparison of bridge beam/girder retrofitting techniques.

Factors	Conventional Technology		Accelerated Construction		
	External Cable Method	CFRP/FRP Sheets	Precast I Girder	Steel Girder	Precast Bulb Tee Girders
Construction Method	High strength tendons are installed drilling hole through beam	Bonded CFRP Plates, External post tensioning with CFRP	Rapid bridge replacement using SPMT	Rapid bridge replacement using SPMT	Rapid bridge replacement using SPMT
Deterioration Factors	cable is exposed to the elements and more susceptible to corrosion	Debonding from substrate, anchorage failure,	Identified as structurally inefficient compared to bulb-tee, Washington, and Colorado girders in terms of cost effectiveness	Welding of connections subject to fatigue	Controlling girder sweep is critical due to slenderness of the section compared to standard girders
Advantages	Improvement of stiffness of beam	Construction less time consuming High strength to weight ratio of material	Standard sections Designers and fabricators are familiar with the sections Performance is well documented	Span range up to 300ft Depth of span ratio(D/S) 0.04 to 0.045 which is smaller than precast prestressed concrete girders	High Performance Concrete (HPC) with 10,000 psi 28-day strength can be used to obtain longer spans
Limitations	Construction may not be possible in specific site or for extremely deteriorated beam	Ultimate strength development may not be possible if the existing concrete quality is poor	Implementation in ABC is only possible with Partial-depth or full depth deck panel High probability of cracking in transfer	Welding of Connections subject to fatigue	Implementation in ABC is only possible with partial-depth or full-depth deck panels.

Note: This table summarizes technical information from several sources (Aktan and Attanayake 2013) (Wipf et al. 2004), (FHWA 2011).

Bridge Deck

The conventional construction technique for bridge deck repair includes overlays with bitumen and patching with normal strength concrete or epoxy. Though easy and low-cost alternatives, bitumen overlays wear out very quickly without frequent maintenance. There are more durable materials to be considered for deck overlays (e.g. Ultra-High-Performance Concrete (UHPC), Latex Modified Concrete) (Khayat and Valipour 2018). For bridge deck replacement, full depth precast deck panel, partial depth precast deck panels are frequently used in ABC projects. The precast decks are joined by deck closure pours or connections, and UHPC is an alternative for this purpose. UHPC builds a composite strong connection between the deck panel and the girder.

Table 5 shows a comparison of the conventional and accelerated construction techniques for bridge deck repair or replacement.

Table 5. Comparison of bridge deck retrofitting techniques.

Factors	Conventional Technology	Accelerated Construction		
	Overlay	Full Depth Precast Deck Panels	Partial Depth Precast deck panels	Deck Closure Pours/connections
Construction Method	Bitumen or Normal strength concrete overlay	Length varies from 8 ft to 16 ft Width varies from 24 ft to 40 ft Nominal thickness 8.5 inch	Thickness 3.5 inch(typical)	Ultra-high-performance concrete is used

Table 5. Comparison of bridge deck retrofitting techniques (continued).

Factors	Conventional Technology	Accelerated Construction		
	Overlay	Full Depth Precast Deck Panels	Partial Depth Precast deck panels	Deck Closure Pours/connections
Deterioration factors	Without waterproofing material the overlay may trap moisture	Tighter tolerances and quality assurance are required during the fabrication process	Panels are typically fragile	Lack of proper curing
Advantages	Easy and quick to install	Relatively fast construction, as overlay is not required	Partial-depth panels can improve work-zone safety and construction speed. Fabrication and handling is simple compared to full-depth deck panels	Full Moment Transfer No- post-tensioning required Low-permeability
Limitations	Cracks and pitting issues are very common	The system consists of too many grouted connections thus make the construction challenging	CIP concrete deck requires extended bridge closure	In high temperature areas shrinkage cracks can form quickly

Note: This table summarizes technical information from several sources (Aktan and Attanayake 2013), (Khayat and Valipour 2018).

Abutment and Footings

The most popular retrofitting technique for abutments and footings are steel and cast in place concrete anchors. Site constraints is an issue for this technique because abutments and footing are difficult to reach, and the strength is not fully restored if the anchor placement is not correct. To overcome this situation, prefabricated abutments or footings are highly effective.

Table 6 shows a comparison of conventional and accelerated construction techniques to repair or replace abutment and footings.

Table 6. Comparison of bridge abutment and footing retrofitting techniques.

Factors Affecting Life Expectancy	Conventional	Accelerated Construction	
	Anchors	Prefabricated Abutments stem/wall	Precast Footings
Construction Method	Ground anchors to stabilize bridge abutments	Precast	Precast
Deterioration Factors	Wrong placement of anchors can lead to failure	The pile cavity forms make the fabrication process challenging.	Cracks due to shipping and handling
Advantages	Effective against seismic load	Abutments precast in segments will alleviate the shipping and handling limitations Large prefabricated elements are advantageous for remote locations where access to the ready-mix concrete is difficult.	Good for small footings Shallow footings have a potential to be supported on piles
Limitations	Concrete casting is sensitive with very margin of error	Tighter tolerances are required for the pile driving operation	A level concrete subbase preparation is necessary, which is an additional operation

Note: This table summarizes technical information from several sources (Aktan and Attanayake 2013), (Modena et. al. 2012)

Choosing a retrofitting method to repair bridge damaged element and restore its capacity is a complex process because there are many factors involved in the decision (Kiani et al. 2018). A performance-based selection criterion based on the structural behavior of the bridge element before and after the retrofit procedure is recommended (FHWA 2019). Axial force, bending, ductility, fatigue, rigid body stability, shear force, torsion responses are influenced by the retrofitting technique.

For instance, if the purpose of retrofitting a column is increasing the load carrying capacity and improving ductility, concrete jacketing can achieve these objectives, however there might be other bridge elements that also need strengthening (e.g., beam or girder which demands increasing the bending moment capacity). In that case, an integrated retrofitting solution or replacement of the damaged elements should be considered. For example, the most popular reason of retrofitting a bridge deck is improving serviceability to reduce vibration and enhancing driving comfort. Overlaying methods address the problem of cracking and deflection due to flexural fatigue and punching shear, but if repairing the bridge deck is not sufficient to restore serviceability and structural capacity, then there are other retrofitting alternatives (e.g., beam addition method, support point addition).

Table 7 shows examples of retrofitting techniques with their main objectives and the expected influence in the structural behavior of the bridge element.

Table 7. Examples of retrofitting techniques and bridge component structural behavior.

Retrofit Objective	Retrofit Technique	Bridge Component Structural Behavior						
		Axial Force	Bending	Ductility	Fatigue	Rigid Body Stability	Shear	Torsion
Bonding	Steel Plate bonding method	•	•		•		•	•
Foundation Retrofitting	Addition of Piles	o	•				•	•
	Ground Anchors					•		
	Improvement of foundation					•		
Jacketing	Continuous fiber reinforced plate jacketing	o	•		•		•	•
	Precast concrete jacketing method	•	•				•	•
	Precast panel jacketing method	•	•				•	•
Overlaying	Lower surface overlaying method	o	•		•		•	•
	Upper surface overlaying method	o	•		•		•	•
Prestressing	External cable method	o	•				•	•
Replacement	Concrete Member Replacement	•	•				•	•
Retrofitting as a structural body	Addition of beams		•				•	•
	Addition of Walls		•				•	•
	Addition of Support Points		•				•	•
Repair of cracks	Fill	o	o		o		o	o
	Injection	o	o		o		o	o

Source: Adapted from (FHWA 2019)

• Commonly used to improve structural behavior
o Potential to improve structural behavior

At present, transportation agencies aim bridges in service that last longer and with less maintenance. Retrofitting techniques in combination with the adoption of innovative materials (e.g., UHPC) are part of the strategy to extend the service life of bridges over 100 years. To compare bridge maintenance and rehabilitation strategies there is a need for material selection criteria based on life expectancy of the bridge element, and cost-effectiveness of the retrofitting technique.

2.3 Concrete Repair Material Selection

A repair project cannot be completed successfully unless the designer identifies the causes of the failure, knowing the advantages and limitations of the retrofitting alternatives under consideration. There are three major challenges in determining the cause of failure. The first challenge is there are not two repair situations alike, and each case should be examined and judged independently. The second challenge is the lack of knowledge about how the performance of existing concrete may influence the effectiveness of the retrofitting technique. It is important that the repair material properties be compatible with the substrate existing concrete. The third challenge is that poor

performance is frequently the result of a combination of number of circumstances, and it may be difficult to identify the main cause of the initial damage.

Mechanical material properties affect repair material interactions with the substrate concrete. Concrete materials may exhibit similar mechanical properties in terms of compressive and tensile strength, although the bonding performance may be different (Haber *et al.* 2018). Conventional concrete is designed with the assumption that the concrete will only carry compression stresses and that all tensile forces are handled by discrete reinforcements such as steel bars. Whereas traditional concrete's tensile strength is frequently ignored in structural design calculations, UHPC's tensile behavior is frequently considered in the design. The tensile response of UHPC is significantly difference from conventional concrete. This tensile capability contributes to the overall structural member capacity and allows slender member proportions in the design solutions.

Abrasion resistance

Abrasion resistance of the repair material should be appropriate for the service environment. Vehicle traffic and abrasion on hydraulic structures, in particular, are aggressive types of abrasion. Wind-blown debris, mechanical contact, and other factors can cause abrasion. Concrete must have a high strength and a low water/cement ratio to withstand abrasion forces. Sound, hard aggregates, and other constituent elements that result in a dense concrete should be used. The standard ASTM tests for abrasion do not predict how long a particular concrete will last. However, they effectively examine how different variables can affect the abrasion resistance of concrete and quality of the surface (ACI 2014).

Bond strength

The ability of two materials to respond as one material is referred to as bond strength or adhesion. A repair material should have enough bond strength to stay attached to the underlying concrete and not separate from it. To assess the bond behavior, standard slant shear tests and splitting tensile tests can be carried out (ACI 2014). The bond strength depends upon factors like surface preparation and treatment technique. Roughness coefficients for treatment techniques are proven to be important in this aspect (ACI 2014). If substantial interface stresses are developed from shrinkage, thermal movement, or other reasons, bond strengths that exceed the tensile strength of either the repair material or the substrate could cause failure in the weaker material.

Creep

Compressive creep may not be relevant when the compressive stresses are minimum. Creep can help to reduce stress caused by shrinkage strain restraints or by thermal movement or live loads.

Coefficient of thermal expansion

It is preferable that coefficient of thermal expansion of the repair material be similar to the substrate concrete, therefore the two materials should behave similarly under temperature variations.

Cohesiveness

The capacity of a repair material to remain intact or not segregate during its application is referred to as cohesiveness. The cohesiveness of a repair material is critical for construction and repair consistency. For example, in vertical and overhead concrete repairs, a more cohesive material can be used in thicker lifts with a low probability of internal separations or debonding before setting.

Elasticity

Elasticity is a material property that measures its capacity to return to its original size and shape after being deformed by a force. Elasticity is crucial for materials that are used to mitigate cracking.

Environmental considerations

The repair material should be appropriate for the service environment. Examples of environmental

and material conditions are air and concrete temperatures, amount of moisture on the substrate concrete surface, relative humidity, wind speed, repair area exposed to sunlight or shade.

Electrical resistivity

Corrosion is a type of electrochemical reaction. If embedded reinforced steel corrosion is a concern, the electrical resistivity of the repair material is critical, especially for concrete replacements. A high or low electrical resistivity may be advantageous depending on the method used to prevent corrosion.

Permeability

The permeability of the repair material or substrate concrete is critical in environments where the repair material or substrate concrete is vulnerable to moisture-related deterioration, such as freezing and thawing damage of saturated concrete, corrosion of embedded reinforcing steel, alkali-aggregate reactions, or sulfate attack. Permeability often diminishes as the cementitious material hydrates or as the carbonation level rises. Therefore, the age of the specimen at the time of the test should always be indicated in the records.

Resistance to freezing and thawing

When exposed to cycles of freezing and thawing in a saturated situation, some repair materials, such as concrete, are subject to damage. Internal pressures can be created by the expansion and migration of water during the freezing and thawing cycles.

Static modulus of elasticity

If the repair is not structural (i.e., it is not intended to share load with the substrate concrete), it is preferable for the repair material to have a lower modulus of elasticity than the substrate concrete, therefore future movements at the repair material-substrate concrete interface can be accommodated. If the volume stability or thermal compatibility of the repair material differ significantly from those of the substrate concrete, a lower modulus is desired.

Significance of compressive, tensile, and flexural strength

Although compressive and flexural strengths rarely limit the performance of repair materials, they can be utilized as broad indicators of material quality. In most cases, these qualities should match or slightly surpass those of the substrate concrete (as measured at the time of the repair, not as originally specified). A repair material with a higher tensile strength may help to prevent cracking caused by shrinkage and thermal contraction.

Viscosity

Low viscosity materials flow more freely than high viscosity materials. Low viscosity compounds are typically utilized to fill fractures and penetrate concrete pores. The technique of measurement (viscometer or rheometer geometry), temperature, and the rate of applied shear strain affect viscosity.

Volume Stability

The volume stability of the repair material has an impact on its compatibility with the substrate concrete. High shear stresses at the interface, debonding from the substrate concrete, and cracking of the repair material can result on significant variations in the volume of the repair material. Repair materials with a lower modulus of elasticity or a faster rate of creep can lessen the stress imposed in the repair material due to restrained contraction and expansion.

Working time

The amount of time available to place the concrete material is influenced by environmental factors that affect the material temperature, mixing ratio, and material mass. As a result, it is critical to determine the working time that should be less than or equal to the initial setting time.

2.3.1 UHPC Material Properties

The material properties of UHPC are evaluated by standard tests, and they vary depending on the mix composition. For example, steel fibers in UHPC mixes contribute to increase the tensile strength capacity, and their orientation influences the mechanical properties. Curing methods and age of loading can also influence creep coefficient, autogenous shrinkage, and total shrinkage (Aarup 2009). Therefore, UHPC mix designs are prepared to meet specific project performance standards for the bridge element. One common application of UHPC in ABC projects is connecting precast elements, and the mix design requires to meet specific properties. Table 8 shows, as an example, the expected range of properties for field cast UHPC (FHWA 2019).

Table 8. Field cast UHPC material properties.

Property	Expected Range
Density	144–157 lb/ft3 (2,300–2,510 kg/m3)
14-day compressive strength	18–22 ksi (125–152 MPa)
Modulus of Elasticity	4,250–8,000 ksi (29–55 GPa)
Poisson's Ratio	0.12–0.2
Direct tension cracking strength	0.8–1.2 ksi (5.5–8.3 MPa)
Direct tension sustained post cracking tensile strength	0.8–1.2 ksi* (5.5–8.3 MPa)
Direct tension bon strength (interface failure)	0.35–0.6 ksi (2.4–4.1 MPa)
Long-term creep coefficient	0.7–1.2
Long-term drying shrinkage	300–1,200 microstrain
Long-term autogenous shrinkage	200–900 microstrain
Chloride ion penetrability	50–500 coulombs
Freeze-thaw resistance	After 600 cycles RDM > 98%
Initial Setting Time	4–10 hour
Final Setting Time	7–24 hour
Static flow	4–10 inches (100–250 mm)
Dynamic flow	7.5–10 inches (190–250 mm)
Alkali-silica reaction	Innocuous

Source: FHWA 2019

Therefore, there are specific UHPC mix proportions required to achieve the mechanical properties established in project performance-based material specifications. Additional examples of UPHPC mix requirements are:

- Water-to-cementitious materials ratio < 0.25.
- Portland cement content ≥ 1,000 lb/yd3 (593 kg/m3).
- Maximum aggregate size ≤ 0.25 times the fiber length.
- Maximum aggregate size ≤ 0.125 inches (3.2 mm).
- Steel fiber–reinforcement geometry, strength, and volume percent.

UHPC Class Materials

Commercially available UHPC is called “UHPC-class”. UHPC has a unique mix design with fly ash, silica fume, and steel fibers. According to ACI, the following mixture constituents have a positive influence in UHPC performance (ACI 2018):

Fly Ash: Reduced water demand; increased air-entraining admixture demand; improved workability; slower rate of reaction; reduced permeability and alkali-aggregate reactivity; improved sulfate resistance; prevent discoloration of concrete.

Silica Fume: Increased water demand; increased air-entraining admixture demand; decreased workability; increased cohesiveness; reduced bleeding; increased plastic shrinkage cracking; darker color; increased compressive and bond strength; increased electrical resistivity; reduced permeability; increased resistance to alkali-silica reaction; sulfate and chemical attack; increased abrasion resistance.

Steel Fibers*:* Reduced slump and workability; tendency for fibers to ball; increased number of smaller-width shrinkage cracks; improved post-cracking ductility; increased compressive, direct tension, shear and torsion, flexural, and flexural fatigue strength; improved toughness and resistance to flexural impact loading.

To develop UHPC-class cost-effective mixes, many research studies have been carried out to find the optimum composition of the mixture constituents (Haber *et al.* 2018), and recent research studies have concluded that UHPC mix is cost-effective.(Wang et al. 2021).

2.3.2 UHPC Mix Design Examples and Applications in ABC projects.

UHPC outlasts conventional concrete in terms of durability because of the thick matrix of the material, discontinuous microstructure, and well-distributed microcracking. UHPC has better resistance to a variety of hazardous gases and liquids, chloride assault, frost action, and freezing and thawing cycles. The greater density of the hydration products and the transition zone between the matrix and the aggregates are responsible for these benefits (Schmidt and Fehling 2005). When compared to conventional concrete, UHPC has a porosity of about 9%, whereas conventional concrete has 15%. (Dugat et al. 1996).

UHPC does not have a standard mix design and, therefore, its mechanical properties vary. FHWA publication titled "Design and Construction of Field-Cast UHPC Connections" (FHWA 2019) recommends that UHPCs with 2% of fiber by volume should reach a compressive strength of at least 14 ksi (97 MPa) prior to being subjected to structural loads. Table 9 shows examples of UHPC mix designs from recent research studies.

Table 9. Mix design of UHPC class materials.

Designation	U-A		U-B		U-C		U-D		U-E	
Mix Design	lb/yd^3	(kg/m^3)	lb/yd^3	(kg/m^3)	lb/yd^3	(kg/m^3)	lb/yd^3	(kg/m^3)	lb/yd^3	(kg/m^3)
Pre-blended dry powders	3503†	(2078)†	3516	(2086)	3600	(2136)	3700	(2195)	3236	(1920)
Water	278	(165)	354	(210)	268	(159)	219	(130)	379	(225)
Chemical admixtures	23	(13.7)	48	(28.7)	preblended*		89††	(53)††	73	(44)
Steel fiber content	277	(126)	88 / 179	(52 / 106)	272	(123.6)	263	(156)	156	(156)
Steel Fiber										
Tensile strength, ksi (MPa)	160 (1100)‡		≥305 (2100) / ≥305 (2100)		348 (2400)		399 (3750)		399 (3750)	
Length, in (mm)	1.18 (30)‡		0.5 (13) / 0.79 (20)		0.5 (13)		0.5 (13)		0.5 (13)	
Diameter, in (mm)	0.022 (0.55)‡		0.012 (0.3) / 0.012 (0.3)		0.012 (0.3)		0.008 (0.2)		0.008 (0.2)	

† : Not pre-blended but come in as separate ingredients, which include fine silica sand, finely ground quartz flour, Portland cement, and amorphous micro-silica

* : The chemical admixtures were dry powders and pre-blended with other powder ingredients

††: It includes three chemicals, a modified phosphonate plasticizer, a modified polycarboxylate high-range water-reducing admixture, and a non-chloride accelerator

‡ : Fibers were straight with hooked ends and did not have a brass coating

Source: FHWA 2018

Table 10 shows examples of ABC projects with UHPC applications, benefits and lessons learned.

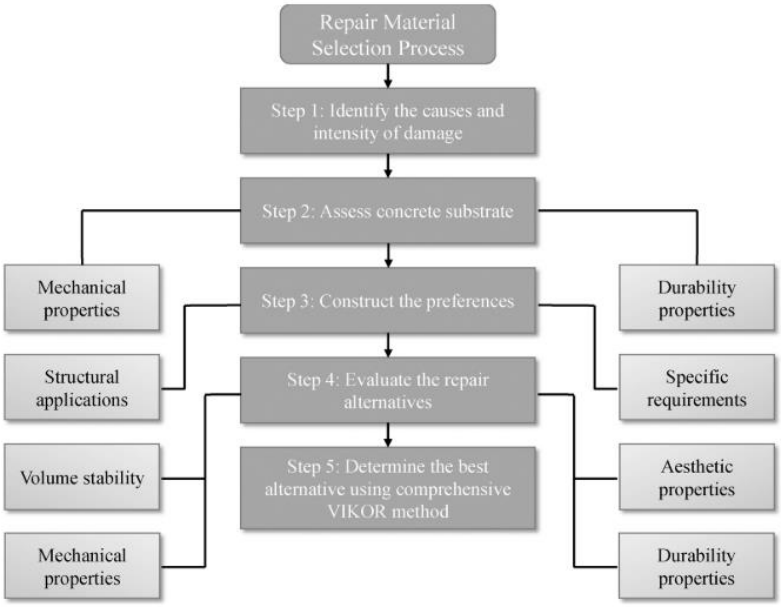
Table 10. Examples of UHPC applications in ABC projects.

Project	Pre-Damage Condition/Purpose	Usage of UHPC	Treatment Benefits	Lessons Learned
Replacement of 1-438 Bridge (2017)	Corrosion and soil instability	Longitudinal connections between the precast adjacent box beams UHPC deck overlay	UHPC's high strength, fast cure time, strong bond, extremely low porosity, discontinuous pore structure, and long-term durability make it superior to conventional concretes and grouts. Effectively waterproofs the entire underlying superstructure, thus protecting it from water and chloride intrusion.	Overlay prone to shrinkage cracks. A UHPC overlay system may not be ideal for bridges with high pedestrian traffic or where pedestrians might be walking barefoot, since the exposed fibers result in a walking surface finish that is rougher, at least initially, than a traditional cast-in-place deck.
The Pulaski Skyway (2017)	Traffic demands were so high, the structure could not be completely closed during the deck replacement.	Precast concrete deck panels connected with ultra-high-performance concrete (UHPC).	The choice of UHPC for the panel connections made the connections the strongest and most durable part of the deck, as opposed to the weak link typically associated with connections between precast concrete elements.	Formwork for UHPC must be watertight. Transporting freshly mixed UHPC by pumping can be problematic, especially in warmer weather. Maintaining a minimum curing temperature of 50°F (60°F preferred) during winter placement of UHPC is critical to avoid having the steel fibers settle toward the bottom and become segregated.
Bridge 7345 New Mexico (2017)	The existing deck and superstructure (5 simple spans) were deteriorated.	The deck closure joints were poured with ultra-high-performance concrete (UHPC)	-	UHPC is sensitive to temperature. Need to substitute ice cubes for water; it is wise to let it not get above 80 degrees. If have lower temperatures, will not start to cure. A definite UHPC pouring plan is needed from contractors
Hanging Rock Creek Bridge (2018)	Longitudinal Shear keys deteriorated	-	UHPC fully develops the transverse deck rebar that is exposed in the connection, resulting in a structurally continuous top slab across the full bridge width. This eliminates the need for longitudinal post-tensioning, which simplifies and accelerates construction.	Before placement of UHPC, the interface surface needs to be roughened to expose the aggregate.

Note: This table summarizes technical information from the ABC-UTC Database of Case studies.

2.4 Guidelines for the Selection of Retrofitting Concrete Materials

Guidelines with a step-by-step framework to assist in the selection of retrofitting concrete materials for ABC projects is provided in this section. Figure 4 shows a flowchart used as a reference to develop the framework. The steps followed in the flowchart to select the optimum concrete material for repair are based on Multi Criteria Decision Making (MCDM). MCDM requires setting targets with weights for durability and mechanical properties expected to be achieved by the concrete material (Kiani et al. 2018).



Source: (Kiani et al. 2018)

Figure 4. Process for selecting concrete repair materials.

The framework proposed to select retrofitting materials for ABC projects consists of seven steps as shown in Figure 5.

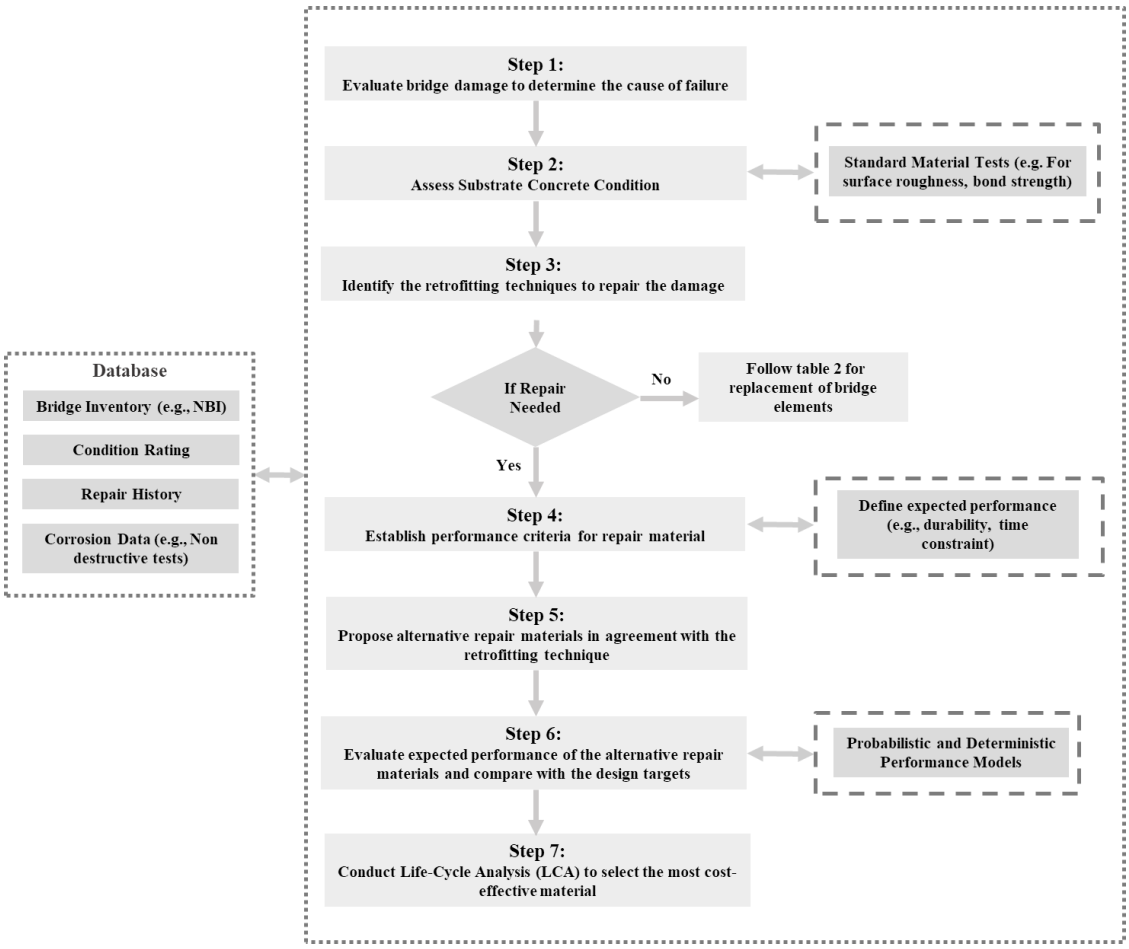
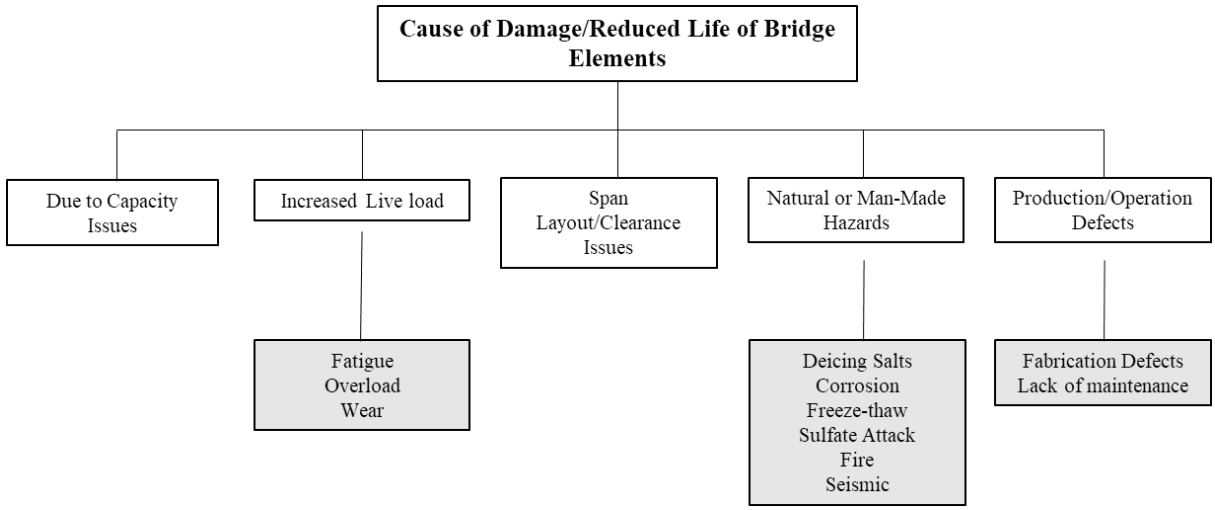


Figure 5. Process diagram to select retrofitting materials for ABC projects.

A description of the steps in the process diagram follows.

Step 1: Evaluate bridge damage to determine the cause of failure.
To choose repair material, at first the existing bridge condition should be examined to determine the cause of failure. The International Concrete Repair Institute (ICRI) guideline states the possible failure characteristics of concrete structures (ICRI 2015). For example, if loss of bond and delamination of concrete occurs, the tensile bond of the repair system may be greater than the tensile strength of the substrate concrete (ICRI 2015). Concrete failure can be caused for several factors including micro-cracking in the cement matrix, debonding from other elements or previous repair, corrosion of steel. Bridge elements can also fail by concrete shear or flexural mechanisms. Concrete shear failures occur in a more brittle manner than flexural failure. Figure 6 summarizes common causes of damage that reduced the service life of bridge elements (Azizinamini et. al. 2013).



Source: "Design guide for bridges for service life" (Azizinamini et al. 2013)
Figure 6. Factors affecting the service life of bridge elements.

Concrete failures are triggered by bridge stressors and the most common are natural and man-made hazards including harsh thermal temperature, coastal climates, and chemical attacks to the material. In addition, significant increases in corridor traffic demand, changes in land use and urban planning development, might eventually cause a bridge's functional inability to provide the expected level of service, forcing bridge replacement or widening. Future operating demands should be also considered in the evaluation of retrofitting alternatives. Therefore, it is necessary to collect accurate and extensive data to operate and manage bridges. Bridge inspections conducted regularly are required to maintain the data current. Bridge inspection methods include routine inspection, in-depth inspection, underwater inspection, and damage inspection (AASHTO 2018).

Step 2: Assess substrate concrete condition.
To ensure proper bonding between the repair material and substrate concrete, it is necessary to evaluate the condition of damaged areas in the concrete and type of defects. There are several methods to evaluate damage and investigate defects in concrete structures. Strength, sonic, ultrasonic, magnetic, electrical, nuclear, thermography, radar, and radiography are methods used to evaluate existing concrete structures. Mechanical sonic pulse-velocity methods have been used for concrete for many years. Rebound and penetration tests measure the hardness of concrete, and they are used to determine the strength of concrete (AASHTO 2018).

It is also important to evaluate the bond strength which strongly correlates with the surface roughness of the substrate concrete. Machine learning techniques in combination with digital image processing

techniques like terrestrial laser scanning can also be used to evaluate surface roughness of concrete substrate. Besides, bi-surface shear, push-off, and slant shear tests could be performed to evaluate the bond strength of UHPC overlays with substrate concrete (e.g., Normal strength concrete, High performance concrete) (Valikhani et al. 2019).

Step 3: Identify the retrofitting techniques to repair the damage.
Depending on the type and severity of the damage, a decision is made about retrofitting technique alternatives to repair or replace the bridge element. Replacement may increase costs, but it is inevitable if structure failure is imminent after few years.

ABC techniques used precast elements and the decision whether is feasible or not should be made. FHWA decision-making criteria, described in section 2.1 of this Chapter, can be followed for this purpose (FHWA 2015). ABC-UTC project "*Development of Guide for Selection of Substructure for ABC Projects*" proposes a framework to select substructure and foundation elements for ABC projects with guidelines for evaluation, design, retrofitting, and construction of new bridges or replacement of existing bridges (Mehrabi and Ali 2019).

Step 4: Establish performance criteria for repair material.
Examination of the substrate concrete is crucial to establish performance criteria for the repair concrete material. For example, performance criteria for a deck overlay could depend on the pattern of cracking, spalling unrelated to reinforced steel corrosion, and any other signs of concrete deterioration. Strength considerations and climate conditions also influences the concrete mix design for repair. Alkali-Silica Reaction (ASR) is a concern for long term concrete durability.

For ABC projects, reduced time of construction and durability are critical performance criteria. The shipping and handling of precast concrete elements facilitates faster construction. A technique to reduce weight is to incorporate voids in the precast concrete elements, which will be filled in situ with concrete. The use of corrugated steel pipe to create voids is widespread, although lightweight concrete can also be used to reduce the weight of pre-cast concrete elements.
The new generation of high-performance concrete mixes offer durability that exceeds the performance of past concrete mixes. For a variety of applications, FHWA has reported promising outcomes of UHPC for ABC projects (e.g., adjacent beam elements, precast full depth deck panel connections) (FHWA 2011). However, corrosion factors affecting UHPC are the same as conventional concrete. UHPC would produce tension fractures under high tensile loading, but cracks are small and closely spaced as compared to the large intermittent cracks found in reinforced conventional concrete.

Table 11 shows factors affecting concrete life expectancy and alternative technology solutions.

Table 11. Technology solutions to extend concrete life-expectancy.

Factor Affecting Life Expectancy	Solution	Advantage	Disadvantage
Freeze and thaw	Good air void system	High resistance to freezing and thawing	Reduction in strength due to extra air
	sound aggregates	durable aggregates	Availability
	Strength of 4000 psi and up	used to overcome stresses	Increased strength makes concrete more brittle
	low w/cm	Reduces infiltration of water	Can produce high strength concrete that is brittle

Table 11. Technology solutions to extend concrete life-expectancy(continued).

Factor Affecting Life Expectancy	Solution	Advantage	Disadvantage
Abrasion and wear	Hard aggregates	Attain high concrete strengths and increased resistance to abrasion and wear	Hard to obtain in some areas
	High-strength concrete	Reduces wearing	Concrete more brittle
	add cover	Provides new surface	Extra weight
Chemical Reactions	nonreactive siliceous aggregates	Reduces Alkali-silica reaction	Hard to obtain in many areas
	Chemical Admixtures	Improved properties	Cost, incompatibility, side effects
	low w/cm	Reduces infiltration of solutions, limits alkalis from outside	Can produce high strength concrete that is brittle
	Blending with limestone	Limits expansion	Reduced skid resistance
Corrosion of Reinforcement	Low permeability	Reduces infiltration of aggressive solutions	Can produce high strength concrete that is brittle
	Membranes and coatings	Reduces infiltration of aggressive solutions	Difficult to apply in the field, wear of traffic
	Sealers for pore lining and blocking	Reduces infiltration of aggressive solutions	Difficult to apply in the field, concrete may be difficult to penetrate
	low w/cm	High strength, low permeability	Excessive cracking, shrinkage
	Low shrinkage	Minimizes cracking	Low water content may adversely affect workability
	Low modulus of elasticity	High deformation, minimizes deck cracking	Reduces concrete stiffness
	Large maximum aggregate size	Less surface area, less water, cement, and paste	It may cause less mix bonding
	Well-graded aggregates	Less paste	Problem when good shape is missing
	Chemical Admixtures	Reduced permeability	Cost, incompatibility, side effects
	Cover	More resistance to penetration of solutions	Wider cracks, extra weight and cost
	Overlays	Creates a low permeability protective layer over the conventional concrete	Difficult to place, expensive, and prone to cracking; proper curing is critical.
Sulphate Attack	Low C3A Contents	Reduces sulphate attack	N/A
	Low w/cm	Reduces infiltration of solutions, limits sulfates from outside	Can produce high strength concrete that is brittle

Source: Adapted from Azizinamini et al. 2013

Step 5: Propose alternative repair materials in agreement with the retrofitting technique.

Concrete repair materials have different mix designs based on unit cost, special admixtures, time of construction, and availability. There are alternative repair materials to concrete. For example, precast girders epoxy injection can be used to repair cracks (NCHRP 2010). In that case, the comparison with UHPC repair options focuses on construction methods, cost effectiveness, and service life expectations. Other alternative repair materials are:

- Fiber reinforced composites (e.g. polymer composites, cement-based composites, and hybrid composites), Carbon Fiber Reinforced polymer (CFRP), Glass Fiber Reinforced Polymer (GFRP), Basalt fiber Reinforced polymer (BFRP)
- Steel (e.g. rods, bars, tendon, plates, strand splice systems, jackets)
- Ultra-High-performance Concrete (UHPC)

- Aluminum Alloy
- Ferrocement
- Shotcrete
- Coatings and sealers

Each repair material has performance advantages and disadvantages. The material usability and shape are important factors. For example, polymer composites may be used as bars, sheets, and wraps. However, complete CFRP wraps cannot be used in T-beams for repair due to the flange (Ghaffary and Moustafa 2020).

Several UHPC mixes are described by FHWA in the study *Properties and behavior of UHPC class materials* (Haber et al. 2018). Typical agency concrete mixes may not always meet the performance specification requirements. A concrete mix producer can create a specialized mix based on the materials available given a set of characteristics. Typically, each plant uses concrete mixes that the agency has developed and approved for the projects (FHWA 2011). Modifications to the concrete mix design can be accepted based on structural tests, and nonproprietary concrete mixes are alternatives.

A base concrete mix design and five additional alternatives are proposed for a bridge deck overlay in the following example. This example is from the research study "High Performance Concrete: Applying Life-Cycle Cost Analysis and Developing Specifications (Jaber 2016).

Base Design Option: Represents the Arizona Department of Transportation (ADOT) Class S concrete with a water/cement ratio of 0.50 and black steel reinforcement.

- *Alternative 1:* Base concrete design adding 20 percent of fly ash by weight of cementitious material.
- *Alternative 2:* Base concrete design adding epoxy steel reinforcement.
- *Alternative 3:* Base concrete design adding 20 percent of fly ash by weight of cementitious material and epoxy steel reinforcement.
- *Alternative 4:* HPC (the base with water/cementitious ratio of 0.40, 20 percent fly ash by weight of cementitious material, silica fume content of 6 percent by weight of cement) and epoxy steel reinforcement.
- *Alternative 5:* HPC (the base with water/cementitious ratio of 0.40, 20 percent fly ash by weight of cementitious material, silica fume content of 6 percent by weight of cement) and stainless-steel reinforcement.

The expected performance of the alternatives is evaluated to finally select the repair material.

Step 6: Evaluate expected performance of the alternative repair materials and compare with the design targets.

Both qualitative and quantitative data should be analyzed while choosing a retrofitting material. Multi Criteria Decision Making (MCDM) (e.g., AHP, Electre, TOPSIS, VIKOR, Simple Additive Weighting, Complex proportional Assessment (CORPAS) models can be used to evaluate expected material performance. Some MCDM methods establish a target criterion (e.g., VIKOR), and the alternative solutions are ranked based on proximity to the ideal answer (Kiani et al. 2018).

Accelerated test methods are also applied at the site to evaluate materials. An example is the concrete maturity accelerated method that is used to evaluate concrete strength. Although, this method does not normally replace the concrete cylinder test requirement, it aids to determine when to test the cylinders (FHWA 2011). ASTM C684 mentions three procedures for concrete accelerated tests: warm water method, boiling water method, and autogenous curing. These tests are used to predict the concrete strength at 28 days (ASTM 1979).

Performance modeling approaches can be deterministic or probabilistic. Sulphate attack, carbonation, chloride ingress are common deterioration factors and their effects on concrete must be modeled. Based on corrosion model, a service life design framework for closure joints was developed by researchers at the ABC-UTC (Jaberi Jahromi et al. 2020). In this study, the factors that influence the service life of closure joints along with the mode of failures were identified. Joint detail options were provided for conventional concrete and UHPC. A life cycle cost analysis was conducted using Life 365 software for options with normal strength concrete. However, there is no performance model for UHPC overlays in the software, and the life expectancy was taken from a reference. As a result, the framework identifies factors that affect the service life of closure joints and performs life cycle cost analysis to select the repair material.

Another approach is to use National Bridge Inventory (NBI) ratings to evaluate and project the bridge condition to determine the need over time for maintenance, rehabilitation, or replacement (NCHRP 2012). Condition inspections are conducted periodically to report the NBI with 9 being excellent condition and 0 absolute failure. A bridge is considered structurally poor if at least one of its components has a condition rating of 4 or less. The condition ratings are projected to identify rehabilitation needs.

Step 7: Conduct Life-Cycle Cost Analysis (LCCA) to select the most cost-effective material.
Cost effectiveness depends on several factors including material cost, mobility cost, cost for installation, time constraints, and others Agency and user costs should be considered in the LCCA of the alternative materials under consideration. The cost of UHPC mixes is different from conventional concrete, and deterministic approaches use the mean or most likely cost values for LCCA (Khayat and Valipour 2018). The material that meets technical criteria with the lowest life cycle cost should be finally selected. The LCCA process to select the most cost-effective material and retrofitting technique is presented in Chapter 4.

There are software tools to conduct LCCA, but they do not include UHPC. For example, “Life 365” is a software that evaluates concrete materials adopting a corrosion model and performing LCCA. Life 365 has been used for deck overlay material selection (Jaber 2016). However, UHPC is not included in this software.

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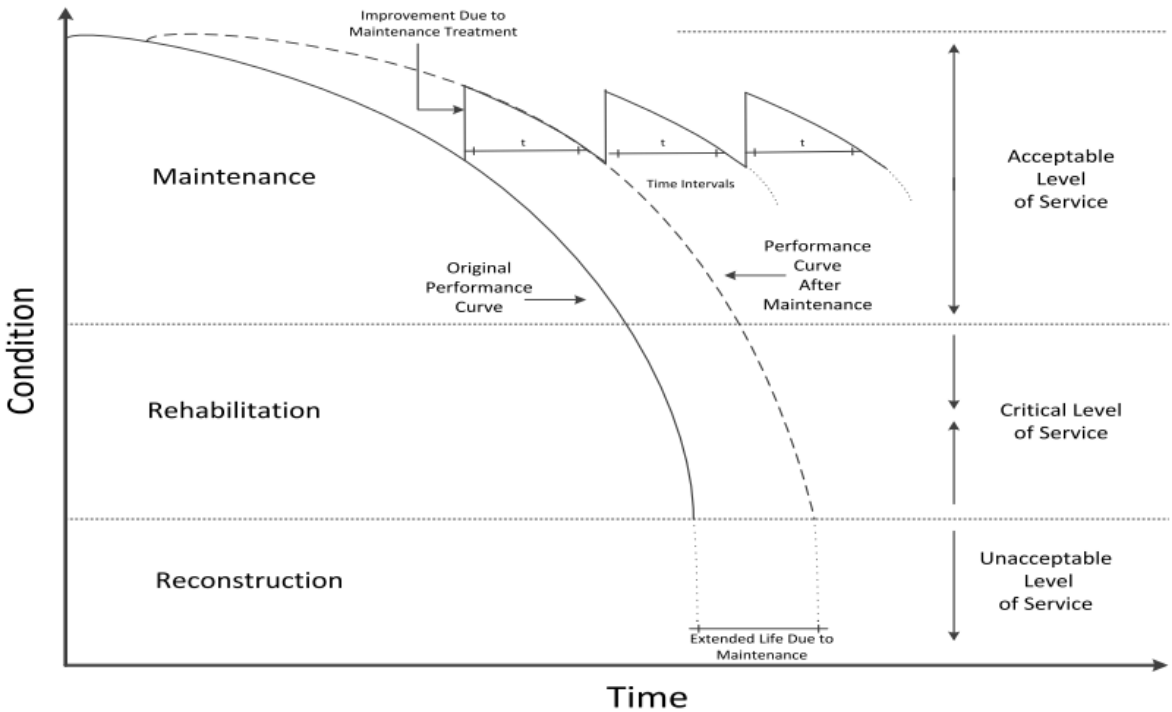
CHAPTER 3: DEVELOPMENT OF LIFE EXPECTANCY AND COST-EFFECTIVENESS MODELS FOR UHPC BRIDGE RETROFITTING TECHNIQUES

This Chapter summarizes deterministic and probabilistic life expectancy models for concrete bridges. Techniques to develop performance models for their application in life cycle cost analysis of UHPC bridge retrofitting techniques are described, and a review of software tools for bridge management is presented. Most of the life expectancy models are based on empirical data (e.g., condition rating), and there are limited records of UHPC performance over time. Therefore, hybrid models are discussed, and a chloride corrosion-based model combined with Monte Carlo simulation is explained as a method to simulate the life expectancy of UHPC.

3.1 Summary of Life Expectancy Models for Concrete Bridges

Bridge deterioration models are broadly classified into deterministic and probabilistic. Both types of models are developed to predict bridge performance to better plan the timing of maintenance and rehabilitation treatments. This section summarizes performance models for network and project management.

The life expectancy of bridge components can be defined in terms of its physical condition and functional performance. Physical condition deteriorates over time until structural failure or collapse, and service life refers to the time until a bridge component can no longer perform its function. Physical condition and functional performance are related, and both should be considered to define life expectancy. Maintenance and rehabilitation treatments can extend life expectancy by improving the physical condition to preserve or restore functional performance. For example, a structural overlay on a bridge deck can extend the physical life, while widening a small bridge can extend its functional life. Figure 7 shows a general condition performance curve over time. The performance curve is adjusted over time due to maintenance treatments applied to preserve the level of service while extending the life expectancy.



Source: Chang et al. 2016

Figure 7. Example of performance curve of asset life.

Life expectancy parameters included in the models for concrete structures are usually climatic conditions (e.g. freeze index and cumulative precipitation), geometrics (e.g., span length and number

of spans), age (overall and since last treatment), construction technique, wearing surface type, bond strength of overlay with bridge deck, highway functional class, repair history, deck area and percent distressed area based on spalling or delamination, traffic characteristics (e.g. volume, wheel location, cumulative truck loads) (Testa and Yanev 2002; Sinha et al. 2005; and Chang and Garvin 2006). Another specific parameter affecting life expectancy is chloride diffusion in concrete locate in the proximity of marine areas or due to the application of deicing salts.

There are tools developed for bridge management to predict the behavior of bridge structures. The tools generally address life expectancy using probabilistic analysis. Some of the tools for bridge management and optimization are AASHTOware Bridge Management, Life-365 and Bridge Lifecycle Cost Analysis (BLCCA). In addition, the Long-term Bridge Performance (LTBP) database has a huge compilation of data of numerous bridge attributes. Table 12 summarized some of the databases and tools for bridge management and life cycle cost analysis.

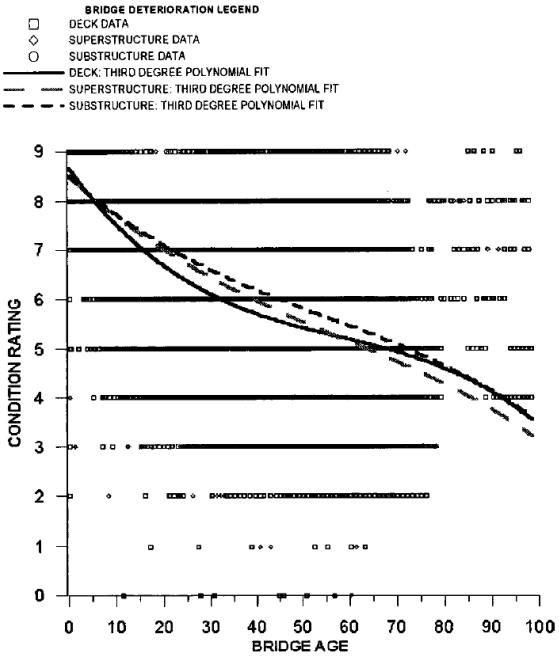
Table 12. Database and tools for bridge management and life cycle cost analysis.

Database/ Tools	Attributes
AASHTOware Bridge Management (formerly Pontis), AASHTO	Stores bridge inspection and inventory data (at the bridge element level), tracks preservation and maintenance, predicts future bridge conditions and incorporates multi-objective optimization analysis at the bridge element level to seek the most cost-effective bridge preservation strategy.
Bridge Lifecycle Cost Analysis (BLCCA), NCHRP	Uses National Bridge Inventory General Condition Rating. Lifecycle analysis for bridges, bridges including agency and user costs. Agency costs from routine maintenance, rehabilitation, and element/ bridge replacement. User costs from detour, crash, and bridge work.
Life-365	Life cycle cost analysis tool for concrete structures. Deterioration model is based on chloride diffusion. User input for concrete mix, agency and user costs.
Long- Term Bridge Performance (LTBP) Program by FHWA	Store bridge performance data. Condition rating, bridge structure and materials data and many attributes are stored in the LTBP.
National Bridge Investment Analysis System (NBIAS), FHWA	Uses national bridge data from the NBI database and offers bridge element–level maintenance simulation capabilities. Forecasts more than 200 measures of bridge performance (including Bridge Health Index, Sufficiency Rating, and Structurally Deficient/Functionally Obsolete Status) over a multi-year period for a range of budgeting levels towards the objective of minimizing lifecycle costs.
Project Level Analysis Tool (PLAT), FDOT	Excel spreadsheet model to complement Pontis (now AASHTOware Bridge Management) at the project level. Dashboard style, uses diminishing marginal returns and incremental benefit/cost. Reports bridge performance, treatment needs, and allocated funding.

Source: Adapted from (Chang et al. 2016)

3.1.1 Deterministic Models

Deterministic models used closed-form equations to model deterioration of bridge condition as a function of age and other explanatory variables. Deterministic models are generally based on an empirical relationship between two or more variables that affect the bridge condition with one dependent variable and one or more independent variables. Some research studies recommended a polynomial curve to predict the condition state of concrete bridges as a function of age as shown in Figure 8 (Bolukbasi et al. 2004; Lu et. al. 2016).



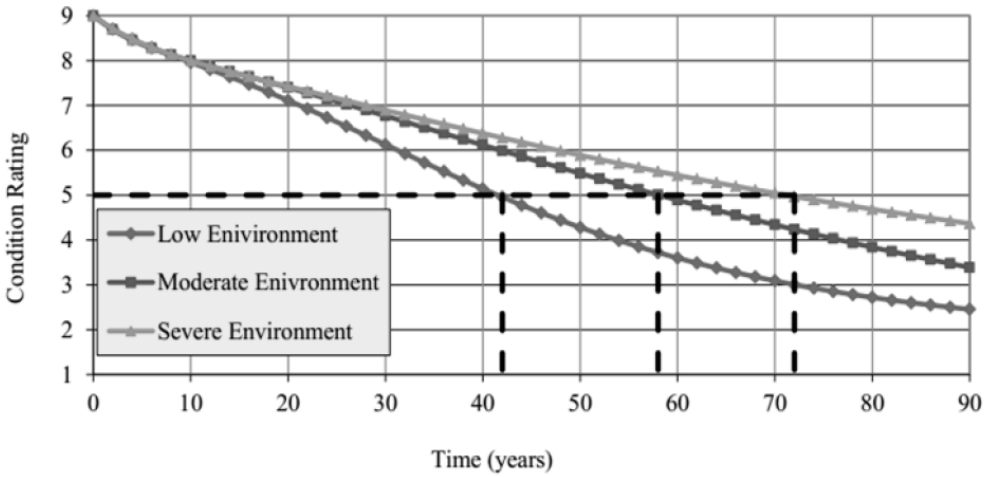
Source: Bolukbasi et al. 2004

Figure 8. Fitting polynomial curve with condition rating of bridges.

The following example is a polynomial equation developed for concrete pier. The output of the equation is the condition rating (CR) as a function of the bridge age. (Agrawal and Kawaguchi 2009).

$$CR = 7 - 0.0486218\ T - 0.001326\ T^2 + 0.0000012\ T^3$$

Deterministic deterioration curves can be further categorized by other factors. Figure 9 shows examples of deterioration curves for concrete decks to predict the National Bridge Index (NBI) in three categories. Low environment refers to an average daily traffic (ADT) below than 1,000 and Average Daily Truck Traffic (ADTT) above 100. Moderate environment refers to an ADT above 1,000 but below 5,000, and ADTT above 100 but below 500. Severe environment refers to an ADT above 5,000, and ADTT below 500 (Morcous 2011).



Source: Morcous 2011

Figure 9. Deterministic deterioration curves for concrete bridge decks using NBI rating.

It is recommended to use models that account for distinct asset attributes (e.g., structural data, environmental, traffic). The viability of this approach depends by whether data are available for these attributes. For example, certain attributes can be useful to predict the life expectancy of bridges

by superstructure type. The attribute could be as simple as the length of the structure. There are two approaches for the development of these models.

Partitioning

Partitioning the attributes of data involves dividing the data set into groups according to one or more classification variables. Table 13 shows an example of culverts data classified by districts (D1, D2 and D3). The average age for culvert replacement can be estimated for each group of culverts in each district with an equation developed using regression analysis.

Table 13. Example of data partitioning for culvert replacement age based on district.

List of culverts with age at replacement				
District name	Culvert identifier	Replace-ment age	Deviation from avg	Square of deviation
District	**CulvertID**	**ReplAge**	**Deviation**	**SqDev**
D1	195451	55	4.25	18.0625
D1	185701	52	1.25	1.5625
D1	137132	47	-3.75	14.0625
D1	194845	49	-1.75	3.0625
D2	268014	42	-1.50	2.2500
D2	205563	47	3.50	12.2500
D2	261619	41	-2.50	6.2500
D2	275579	48	4.50	20.2500
D2	226692	39	-4.50	20.2500
D2	278272	44	0.50	0.2500
D3	352904	46	5.40	29.1600
D3	372275	41	0.40	0.1600
D3	326486	37	-3.60	12.9600
D3	306439	39	-1.60	2.5600
D3	314958	40	-0.60	0.3600

Source: NCHRP 2012

Linear and non-linear models

This approach models life expectancy as a function of one or more explanatory variables. Regression analysis is used to develop from simple linear models to sophisticated non-linear models. In Table 14, the explanatory variables are District 1 and District 2 and the length of the culvert. Although District 3 was an explanatory variable under study, it was not considered in the final equation since only significant variables were included.

Table 14. ***Example of linear regression results for culvert replacement.***

List of culverts with age at replacement								
District name	Culvert identifier	Replace-ment age	1 if D1	1 if D2	Barrel length	Predict age	Devi-ation	Sq of Devn
District	**CulvertID**	**ReplAge**	**Dist1**	**Dist2**	**Length**	**Pred**	**Devn**	**SqDev**
D1	195451	55	1	0	20	52.50	2.50	6.27
D1	185701	52	1	0	36	51.02	0.98	0.95
D1	137132	47	1	0	40	50.68	-3.68	13.56
D1	194845	49	1	0	62	48.80	0.20	0.04
D2	268014	42	0	1	48	45.65	-3.65	13.29
D2	205563	47	0	1	59	44.62	2.38	5.68
D2	261619	41	0	1	86	42.30	-1.30	1.68
D2	275579	48	0	1	77	43.03	4.97	24.69
D2	226692	39	0	1	100	40.99	-1.99	3.95
D2	278272	44	0	1	62	44.42	-0.42	0.18
D3	352904	46	0	0	48	44.80	1.20	1.44
D3	372275	41	0	0	106	39.65	1.35	1.82
D3	326486	37	0	0	86	41.37	-4.37	19.11
D3	306439	39	0	0	120	38.44	0.56	0.32
D3	314958	40	0	0	116	38.74	1.26	1.58

Source: NCHRP 2012

Average and standard deviation of repl age			
District name	Number of culverts	Average age	Population StDev
District	**Count**	**AvgAge**	**PopStDev**
D1	4	50.75	2.28
D2	6	43.50	2.87
D3	5	40.60	2.20

Regression results			
R-squared	0.75		
Variable	Coeffi-cient	Standard error	t-Statistic
Intercept	49.02	3.77	13.01
Dist1	5.22	2.85	1.83
Dist2	0.85	1.97	0.43
Length	-0.09	0.04	-2.38

There are deterministic performance models for concrete bridges to evaluate the influence of loadings and vibrations, extreme weather conditions, presence of chlorides in de-icing salts and freeze and thaw cycles, and air borne chlorides in marine environments. These factors contribute to reinforced concrete deterioration, predominantly due to steel corrosion. Reinforced concrete deterioration can be explained with a corrosion model. Most of the effects of corrosion are modeled through equations with parameters changing with time. The advantage of using these deterministic models is that provides a quantitative assessment of the physical condition of the bridge based on material properties, stress conditions, and structural behavior.

Changes in the corrosion rate over time depend on climatic parameters. The two parameters that influence most in the corrosion rate are the moisture content in concrete pores and temperature. The moisture depends on the atmospheric relative humidity and rain intensity. It may be expected that if concrete is maintained in chambers with constant humidity and temperature, the corrosion rate would remain constant as well. However, corrosion is a dynamic process since the formation of oxides is continuous as well as the extension or increase in depth of the local attack, and for this reason the concrete condition changes over time.

Modeling the deterioration of concrete due to chloride ingress

It is assumed that, in a de-icing salt environment, diffusion is the leading transport mechanism in concrete once the chloride has passed the surface zone. With respect to chloride-induced corrosion, the thickness and permeability of the concrete cover determines the structure's resistance to chloride ingress. Most often the quality of the concrete is expressed in terms of a diffusion coefficient. Most common equations to describe chloride ingress into concrete are based on Fick's second law of diffusion called the error function solution.

The equation for chloride content diffusion has different forms and the most common equation for chloride concentration C(x,t) at depth (x) and time (t) is (Morcous and Lounis 2007) :

$$C(x,t) = C_s - (C_s - C_i) \cdot erf\left(\frac{x}{2\sqrt{D_a \cdot t}}\right)$$

Where:

C (*x*,*t*) = chloride content at depth x and time t, [%m/m cement];

C_s = chloride content at exposed concrete surface, [%m/m cement];

C_i = initial chloride content resulting from contamination of the concrete mix ingredients, [%m/m cement];

x = distance to the concrete surface exposed to chlorides, [mm];

Da = apparent chloride diffusion coefficient, [mm 2 /yr];

erf error function.

Crank's closed-form solution of Fick's second law of diffusion for a semi-infinite medium is (Morcous and Lounis 2007):

$$C(x,t) = C_s\left[1 - erf\left(\frac{x}{2\sqrt{Dt}}\right)\right]$$

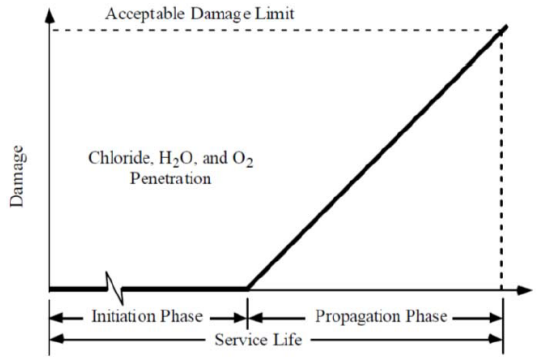
Where:

Cs = Surface chloride concentration

D =Diffusion coefficient of chlorides

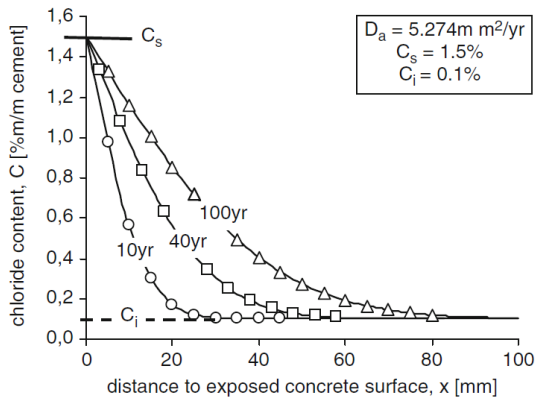
The Initiation Phase
The initiation phase is the time that it takes for chloride ions to pass through the concrete cover to the reinforced steel, reach a threshold concentration, and begin the corrosion process. Therefore, it is the time period from the initial exposure to chlorides until the onset of corrosion as shown in Figure 10.

Propagation Stage:
Post-corrosion stage that corresponds to the initiation of damage (e.g. cracking, delamination, spalling). The propagation process begins after the initiation phase and ends when the structure is no longer structurally sound to perform its intended function. In most cases, the initial period is much longer than the propagation period. The model is based on chloride concrete diffusivity, which controls the time it takes for chloride ions to travel through concrete and reach the reinforcing steel. By analyzing the chloride ingress, a chloride diffusion coefficient and surface chloride concentration can be obtained from core samples extracted the deteriorated bridge. Then, both parameters can be derived by fitting Fick's second law of diffusion to the observed chloride profile.



Source: Tuutti (1982)
Figure 10. Service life prediction model for reinforced concrete bridges exposed to chloride.

An acceptance criterion should be established to model the deterioration due to chloride diffusion. With respect to chloride ingress, it is generally agreed that for new concrete structures, the end of service life is reached when depassivation of the embedded steel reinforcement occurs. This implies that service life is defined by the time when the reinforced steel reaches the critical chloride content. Figure 11 shows theoretical chloride profiles calculated using a constant diffusion coefficient. If the surface chloride concentration (Cs) and initial chloride (C_i) are known, a chloride profile can be developed where x is the distance to the exposed concrete surface (GAAL 2004).



Source: GAAL 2004
Figure 11. Theoretical chloride profiles calculated according to Fick's law of diffusion using a constant D_a.

The following section describes important aspects of the parameters used in the chloride diffusion equation.

Critical chloride concentration
As explained in previous research, the following chloride threshold concentration values can be obtained (GAAL 2004):

- The chloride level at the depth of the steel, which initiates corrosion, independent from any damage to the structure.
- The chloride level at the depth of the steel that results in developing damage (Cracking, spalling) due to corroding reinforcement.
- The chloride level at the depth of the steel that, within the intended life span of the structure, results in corrosion and causes spalling and cracking.

Therefore, the critical chloride concentration is the concentration value at which corrosion initiates independent from any damage to the structure. The threshold for the chloride concentration level can be interpret as a mean value that reflects the probability of corrosion as chloride concentrations increases over time. Critical chloride concentration holds a lognormal distribution (Morcous and Lounis 2007), and there is not a universal chloride threshold value for corrosion initiation.

Time dependent diffusion coefficient
The chloride diffusion coefficient is used to measure the chloride penetration at different condition states. Examples of diffusion coefficient values from previous research are:

- 2×10^{-15} m^2/second in steady-state conditions and 3×10^{-11} m^2/second in non-steady-state conditions (Andrade et al. 2003)
- 1.3×10^{-13} m^2/second at 28 days (Thomas 2012)
- 2.3×10^{-13} m^2/second in a non-steady-state condition (Thomas 2012)

It was commonly assumed that the diffusion coefficient for concrete remains constant over time, although observations suggested otherwise. The resistance to chloride diffusion is increased over time because the concrete pores are reduced, and the equation to model the time dependency of the diffusion coefficient is (Maage et al. 1996):

$$D_{cl}(t) = D_{cl;0}\left(\frac{t_0}{t}\right)^n$$

Where:

Dcl(t) = Time dependent diffusion coefficient

Dcl;0 = Reference chloride diffusion coefficient

t0 = reference period

t = time of exposure

n = Aging coefficient (0<n<1)

Influence of temperature differences on chloride ingress
The ingress of chlorides is faster at high concrete temperatures than at low temperatures. Increased ion transport is usually associated with a higher diffusion coefficient. Many studies have been conducted, although findings vary significantly ranging from a 100% increase at a 50°C temperature difference (Andrade 2003) to a 20% increase at a 60°C temperature difference (Jooss et al. 2002)

3.1.2 Probabilistic Models

Probabilistic performance models address the uncertainty of the factors that influence the bridge deterioration process. Probabilistic performance models are classified as state-based or time-based. In state-based probabilistic models, the deterioration process is modeled by the probability of transition to move from one condition state to another in a discrete time interval. Markov chain is the most common renowned state-based model, and it is based on the following assumptions (NCHRP 2012):

- The future condition state of the bridge depends on the current condition state.
- The rates of transition from one condition state to another remain constant throughout the service life.
- The probability of moving from one condition state to another is constant.

Figure 12 shows an example of a transition probability matrix in which each condition state transition is represented by a probability. The example shows that if a bridge component is in condition state 1 this year, then there is a 95.3% chance that it remains in state 1 next year. If there are 100,000 feet of bridge components in state 1, then 95,300 feet remains in state 1 the next year, 4,600 feet in state 2, 100 feet in state 3, and none in states 4 or 5. Probability values in each row sums to 100% (NCHRP 2012).

Bridge condition in future years can be predicted with a Markovian model using a simple matrix multiplication with the equation shown in Figure 12. Where x_j is the probability of being in condition state j at the beginning of the year and y_k is the probability of being in condition state k at the end of the year. P_{jk} is the transition probability from j to k. A bridge can begin in state 1 and predict the average service life using the transition probability matrix. This model can be applied to a bridge network or individual bridge components. If bridge substructure and superstructure condition ratings are available, then it is possible to predict the condition rating of a bridge element (e.g., Bridge Deck).

Markov transition probability matrix					
State	State probability in one year				
Today	1	2	3	4	5
1	95.3	4.6	0.1	0.0	0.0
2	0	93.2	3.9	1.9	1.0
3	0	0	89.4	7.3	3.3
4	0	0	0	82.8	17.2
5	0	0	0	0	100

Probability of state k next year: $y_k = \sum_j x_j p_{jk}$ for all k

j is the condition state this year and x is the fraction in state j
p is the transition probability from j to k

Source: NCHRP 2012

Figure 12. Example of a Markov transition probability matrix.

Markov models provide a qualitative measure of the bridge condition (e.g., good, poor) useful for network level analysis, although for individual projects more details may be needed. For example, the Markov chain model cannot assess the condition of a structure in terms of stresses to evaluate the structural condition (Noortwijk e.t al. 2004). The memoryless assumption of Markov model implies that the rate of deterioration does not increase with age. As an improvement to the Markov model, time-based models are utilized to provide an age-dependent likelihood of failure (NCHRP 2012).

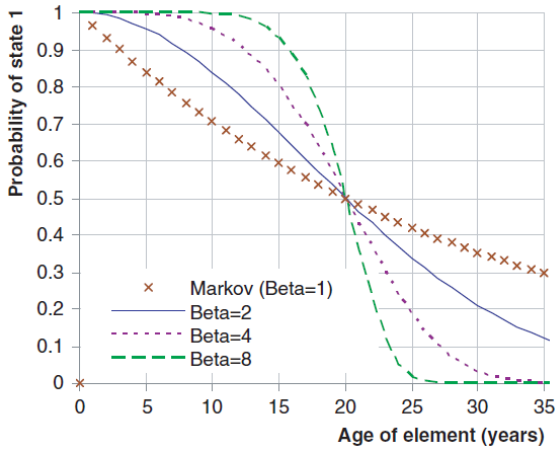
In time-based models, the deterioration process is represented by Weibull, Gamma, and other probability distributions. The probability that a bridge element remains in a particular condition sate is modeled as a random variable. For example, Florida DOT uses a semi-Markov model which incorporates Weibull distributions to model the probability of remaining in a condition state as a function of age (Sobanjo 2011). By applying a Weibull survival probability model, age dependency is integrated to the Markov model. The Weibull probabilistic deterioration curve has the following functional form:

$$y_{1g} = \exp\left(-1.0 \times (g/\alpha)^{\beta}\right)$$

where y_{1g} is the probability of the not-failed state at age g, if there is no maintenance between year 0 and year g; beta is the shaping parameter, which determines the initial slowing effect on deterioration and alpha is the scaling parameter, calculated with this equation:

$$\alpha = \frac{t}{(\ln 2)^{1/\beta}}$$

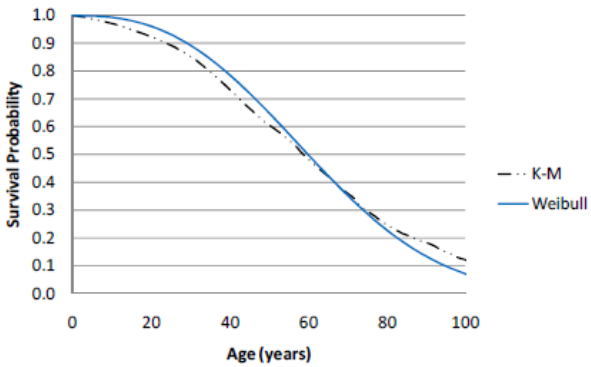
where *t* is the median life expectancy calculated from the Markov model. Figure 13 shows Weibull curves for four beta shaping parameters. The rate of deterioration is low at the beginning of the service life, but it increases over time. The curves intersect at year 20 with a probability of 0.5 because the median transition time is the same in all the scenarios. The beta shaping parameter can be calculated using a statistical process known as "maximum likelihood estimation," which is a structured trial-and-error procedure to find the best fit for beta values (NCHRP 2012). The Beta distribution is recommended as the best to model the remaining service life, particularly after a treatment is applied (Sobanjo 2011).



Source: NCHRP 2012

Figure 13. Examples of Weibull survival probability model.

Probabilistic models can be inferred as covariate and non-covariate models. Examples of data in covariate models are temperature, corrosive soil indicators, geographical classification indicator. Figure 14 shows a Weibull-distributed bridge life covariate model. This model estimates a baseline median life of 60 years with a 90% confidence interval.



Source: NCHRP 2011

Figure 14. Non-parametric validation of Weibull-distributed bridge life covariate model.

Table 15 summarizes bridge performance models previously described in this section. It is important to retrieve the data for a reliable source and analyze the attributes before finally choosing a performance model to estimate life expectancy.

Table 15. Summary of bridge performance models.

Performance Models		Governing Equation/Method	Data	Advantages	Disadvantages
Deterministic	Condition Polynomial curves		Condition rating	Suitable for Project Level Analysis Provides projection of deterioration of bridge elements over its lifespan	Model is costly in terms of data requirements and modeling
	Chloride induced corrosion model (Equation Based)	Fick's Law of diffusion	Chloride diffusion coefficient, critical chloride threshold, cover depth, environmental exposure parameter		
	Carbonation induced corrosion (Equation based)		Carbonation data		
Probabilistic	State Based	Probabilistic analysis using Transition probability matrix (percentage prediction method, regression-based optimization) to predict change of one state to another	Condition rating (deterioration process is modeled through a probability of transition from one state of another in discrete time interval	Provides a framework that accounts for the uncertainty Compatible with existing qualitative/discrete bridge condition rating systems Practical at the network level	Only provides a qualitative prediction of the future condition of bridge element (e.g., excellent, good, fair etc.) Cannot be used to assess the reliability of a structure in terms of strengths and stresses
	Time Based	Models the probability of remaining in a condition state	Condition rating as a function of age, AADT etc.	Time-based models have been used to obtain an age-dependent probability of failure as an enhancement of the markov model Practical at the network level	Interaction between different elements in relation to the structural integrity is ignored

Source: Adapted from (Skrikanth and Arockiasamy 2020)

3.1.3 Techniques to Develop Bridge Performance Models

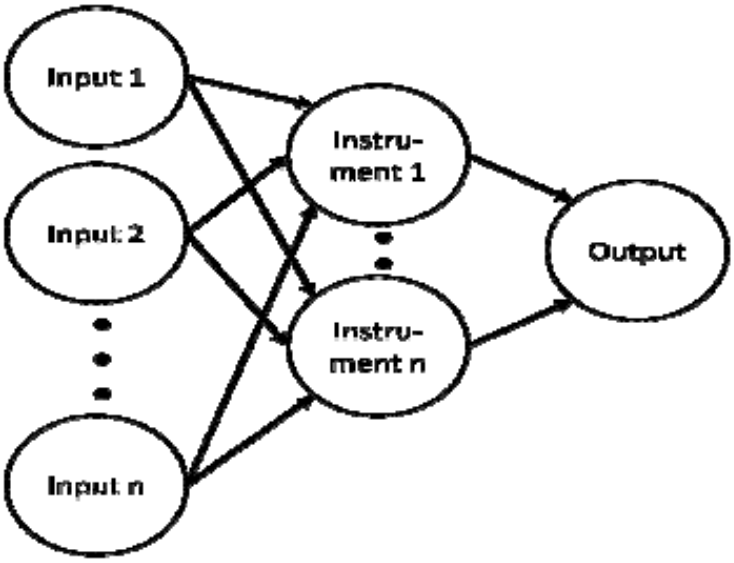
The common techniques to develop bridge performance models are regression, Monte-Carlo, and artificial intelligence.

Regression based techniques

Regression-based techniques are used to develop linear and non-linear models. The output of regression-based techniques are closed-form equations that predicts the condition as a function of independent variables (e.g., AADT, bridge length, asset demolition records, maintenance periods). These are also called explanatory variables that predicts the service life of the entire bridge or a bridge element. A variable is considered a strong contributor to the model if the absolute value is at least 1.5 or 2.0. However, if the variable contributes to the model's intuitive sensitivity or is required for its use, a smaller t-Statistic is acceptable (NCHRP 2012).

Monte Carlo simulation
Monte Carlo is the most widely technique used to address uncertainty in a broad range of applications. Monte Carlo simulation is a dataset upsizing method that can take several parameters into account simultaneously. This technique involves sampling at “random” from the distribution of data inputs to simulate a large number of experiments until a statistically significant distribution of the structure response is generated. Monte Carlo analysis follows a repeated statistical sampling process by which randomly selected values are used to develop a limit state function. A random number based on a given mean and standard deviation is selected in each iteration and used in the limit state function. These random values are described by probability density functions.

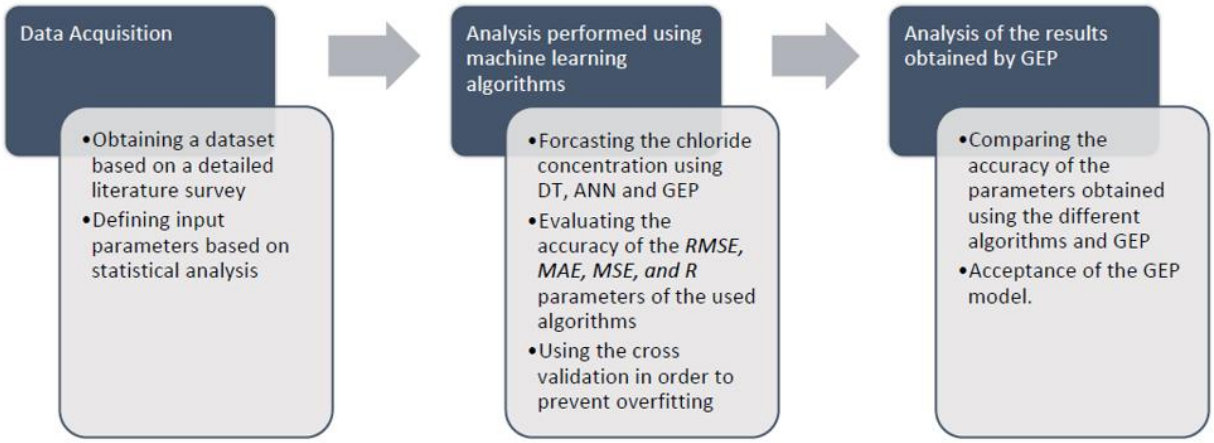
Artificial Intelligence based techniques
Machine learning is a mathematic technique for life prediction that has gained traction among some researchers. This non-linear adaptive model anticipates situations based on what it has been "learned" by identifying patterns or trends from previous observations. An artificial neural network, the most common learning technique, is a non-linear version of the 3-Stage Least Squares regression, in which “instruments” that express relationships with other variables are estimated to predict future occurrences or outputs as shown in Figure 15 (NCHRP 2011).



Source: NCHRP 2011
Figure 15. Example of artificial neural network.

Machine learning models are used to predict surface chloride concentrations in concrete (Ayaz et al. 2021). Machine learning (ML) is an approach to predict the mechanical properties of concrete including the assessment of surface roughness of the substrate concrete condition (Valikhani et. al 2020). ML algorithms are more effective than correlation models due to the fact that several variables are involved in the development to predict surface chloride concentrations instead of one single variable. Artificial neural networks (ANN), decision trees (DT), support vector machines (SVM), random forests (RF), gene expression programming (GEP), and deep learning (DL) are the most common algorithms to analyze the mechanical properties of concrete (Ayaz et al. 2021). A combination of the machine learning techniques “Transfer Learning” and “Data Augmentation” could be used for small datasets (Valikhani et al. 2020).

Figure 16 shows an example of a general framework for the application of artificial intelligence for life expectancy modeling.



Source: Ayaz et al. 2021

Figure 16. Framework for the application of artificial intelligence for life expectancy modeling.

Techniques to develop life expectancy models are summarized in Table 16 with their advantages and disadvantages.

Table 16. Techniques to develop bridge performance models.

Technique	Approach	Data	Advantages	Disadvantages
Regression	Binary Logistic	Bridge structural data, condition rating, AADT, age, type of design, distance from seawater and others.	Simplest approach to predict the future condition of bridges Practicality at the network level.	Neglects uncertainty due to the inherent stochastic nature of deterioration. Disregards interaction between deterioration of different bridge components.
	Two Step Cluster			
Reliability based models	Monte Carlo method	Chloride corrosion model data.	Probabilistic forecast of future deterioration Sensitivity analysis to finds out the governing parameters.	Field data is needed.
Artificial-Intelligence (AI) based models	Expectation maximization (EM) approach	Condition rating, age, design load, maintenance history, length, ADT, Deck area, environment, number of spans, degree of skew and others.	ANN based technique can generate missing condition state data to fill the gaps due to missing inspections. CBR technique can be used to perform "what if" analyses for different maintenance scenarios.	ANN is an approach to artificially generate missing data and it needs complementary tools to utilize generated information for modeling bridge deterioration. The performance of the CBR approach depends on the size of the case library and the adequacy of case description.
	Case -based reasoning (CBR)			
	Genetic algorithms			
	Shuffled frog leaping			
	Particle swarm optimization			
	Artificial Neural Network (ANN)			
	Back Propagation method with multi-Layer Perceptron classifier (BP-MLP			
	Random forest model			

Source: Adapted from (Skrikanth and Arockiasamy 2020)

3.1.4 Selection of Modeling Techniques and Data Attributes

Three dimensions of analysis should be considered when choosing a modeling technique: the first dimension relates to basic asset attributes (e.g., asset class, design/material type); the second dimension relates to the nature of the data (e.g., if the data is cross-sectional, timeseries, or panel; if the dependent variable is discrete or continuous; if sufficient condition-based, age-based); and third dimension relates to the modeling techniques such as if a deterministic or probabilistic model is

preferred; the specific statistical technique that is to be used; and the measure of goodness-of-fit used to validate the model results (NCHRP 2011).

The selection of a method to develop life expectancy models depends on the type of data available. For example, when condition measures are routinely collected as part of an inspection, models can be calibrated. Data needs vary with different models. Data for past replacement actions, past life extension actions, relevant inventory, condition data are adopted in certain models. When the data is not routinely collected or it is of continuous nature, a duration model can be fit to the time at which the threshold value is crossed. Time series of past condition records are used in probabilistic models. The consistency of data collection and processes over time is very important when building the models. Data should represent the population whose life expectancy is desired.

Performance models can be developed from data stored in the National Bridge Inventory (NBI) database. Various performance measures exist in the NBI database that can be used to determine the in-service life. These measures include 'Sufficiency Rating', 'Inventory Rating', 'Structural Evaluation', 'Deck Geometry', 'Bridge Posting', 'Scour Critical Bridges', 'Deck', 'Substructure', or 'Superstructure' (FHWA, 1995).

Bridge inspections are typically conducted biannually, pending special exemptions or critical status, and the NBI database stores the following information:

- Design and geometric parameters of the bridge including span length, skew angle, deck width, material type, superstructure design type and design load.
- Operational conditions such as Average Daily traffic (ADT), age, and highway classification.
- Structural condition of bridges in the states inventory.

Table 17 shows the NBI condition rating criteria from excellent (NBI condition rating = 9) to failed (NBI condition rating = 0). Bridges with condition rating 4 or below are considered deficient.

Table 17. NBI condition rating.

Condition Rating	Condition	Description
9	Excellent	
8	Very Good	No Problems Noted
7	Good	Some Minor Problems
6	Satisfactory	Structural Elements Show some minor Deterioration
5	Fair	All Primary structural Elements are sound but may have minor section loss, cracking, spalling or scour
4	Poor	Advanced section loss, deterioration, spalling or scour
3	Serious	loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete maybe present
2	Critical	Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close bridge until corrective action is taken.
1	Imminent Failure	Major Deterioration or section loss present is critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service
0	Failed	Out of service- beyond corrective action

Source: FHWA 1995

Bridge management systems
Bridge management systems (BMS) are decision-making supporting tools developed to assist agencies formulate maintenance and rehabilitation plans. The BMS database components can include (NCHRP 2012):

- Geo-referencing database (usually integrated with the agency GIS)
- Traffic count database (often integrated with the GIS)
- Crash database (often maintained outside the transportation agency)
- Bridge inventory
- Bridge condition (time series of inspections or surveys for each asset)
- Bridge vulnerability to natural and man-made hazards (time series)
- Climate condition database (often maintained outside the transportation agency)
- Soil characteristics database (often maintained outside the transportation agency)

Climate and Severe Events Data
The National Oceanic and Atmospheric Administration (NOAA) maintains a variety of climate and severe events data, the majority of which can be accessed and downloaded for free (http://ww.ncdc.noaa.gov/oa/mpp/freedata.html). The Natural Resources Conservation Service is in possession of soil data (NRCS). Because of the deterioration of the below-ground components of culverts and bridges, soil attributes such as corrosiveness and frost action potential (rated from no potential to high potential) have a substantial impact on their service life.

3.2 Life Expectancy Software Tools and Databases

Certain bridge elements are built to withstand the most wear and are replaced or retrofitted at regular intervals to protect the larger and more expensive components. Expansion joints, coating systems, deck wearing surfaces, cathodic protection systems, bearings, drainage systems, pile jackets, fenders, and slope protection are examples of protective elements that can expand the lifespan of a bridge. UHPC is commonly used in bridge deck overlays, in column protective shells, and on expansion bridge deck joints. The protective components are crucial to extend the life expectancy.

Software tools have life expectancy models built from different parameters for conventional concrete mixes. Two of these software tools are Life 365 and the Bridge Life Cycle Cost (BLCC) software tool. There are limited historical data regarding the performance of UHPC application in retrofitting applications. Therefore, the models in the software tools are only a reference for the development of life expectancy model for UHCP.

3.2.1 Life 365 Software Tool

Life 365 estimates the life cycle cost of new bridge concrete elements based on the chloride corrosion model. There are three types of elements: slabs and walls (1-D), square column/beam (2-D), and circular columns (2-D). After choosing the bridge element, concrete mix design information is required for the alternatives under analysis. Concrete mix design data includes the water cement ratio, fly ash %, slag %, and silica fume % for Conventional Strength Concrete (CSC) or High-Performance concrete (HPC) with added silica fume and fly ash. The rebar type alternatives are black steel, stainless, or epoxy coated. In the analysis, the life expectancy of the “Base Case” for a normal strength concrete mix is compared to the other alternatives defined for analysis. The base case has pre-defined recommended values for the chloride diffusion parameters.

Table 18 lists the data needed for the Life 365 model to compare alternatives.

Table 18. Data inputs for Life 365.

Structural Data	Chloride Exposure Data	Life Cycle Cost Data	Concrete mix design
Type of structure (e.g., slabs and walls/ column/ beam), Thickness, Reinforcement depth, Area.	Based on Specific US state (e.g., Florida, New York etc.) or Manual entry of maximum chloride concentration according to ASTM C1556, Temperature over the year, Chloride diffusion coefficient (D28 m2/sec)	Base year, Discount rate, Inflation rate, Analysis period, Repair cost, Construction Cost.	Water/cement ratio, fly ash (%), slag (%), silica fume (%), Rebar type, Rebar % vol. concrete, Barriers (Membrane or Sealer), Corrosion Inhibitors.

Then, the analyses process can be divided into four steps:

Step 1: Predicting the time to the onset of corrosion of reinforcing steel or initiation period.
The initiation period is the time for chlorides to penetrate the concrete cover and accumulate in sufficient quantities at the depth of the embedded steel to cause corrosion. It predicts the critical chloride concentration and time to reach the depth of the concrete cover. The model employs a simpler approach based on Fickian diffusion while considering time dependent changes in diffusion. To model the exposure concrete condition, there are three options:

1. To select the values from a built-in database based on the type of structure (e.g. wall/slab, beam, column), the type of exposure (e.g., marine exposure or deicing salts), and the geographic location (e.g. Florida, New York).
2. To establish a maximum chloride concentration as percent of weight of the concrete mix and time to reach this value.
3. To indicate the chloride amount at various depths as mentioned. In accordance with ASTM C1556, chloride profiles can be prepared for different surface chloride concentrations to find the maximum.

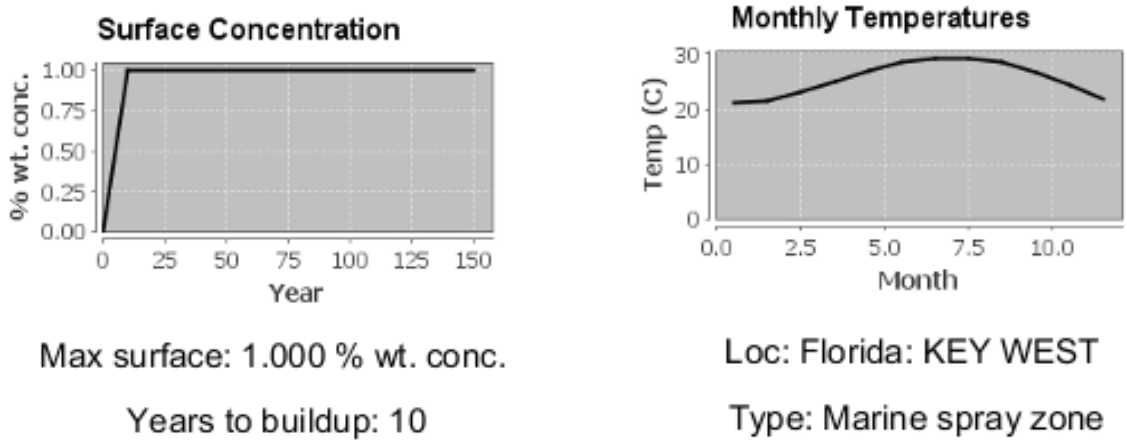
Default temperature profiles are available by geographic locations, although the temperature profile can also be defined. The initiate corrosion time is calculated using the Fick's second law changing the value of D in this equation.

$$\frac{dC}{dt} = D \cdot \frac{d^2C}{dx^2},$$

Where:
C = the chloride content
D = the apparent diffusion coefficient
X = the depth from the exposed surface
T = time

The following time dependent diffusion coefficients are recommended for the case base normal strength concrete equation:
D28 = $1 \times 10^{(-12.06 + 2.40w/cm)}$ (meters-squared per second (m2/s), which is the diffusion coefficient at t = 28 days
n = 0.2
C_t = 0.05 percent (% wt. of concrete)

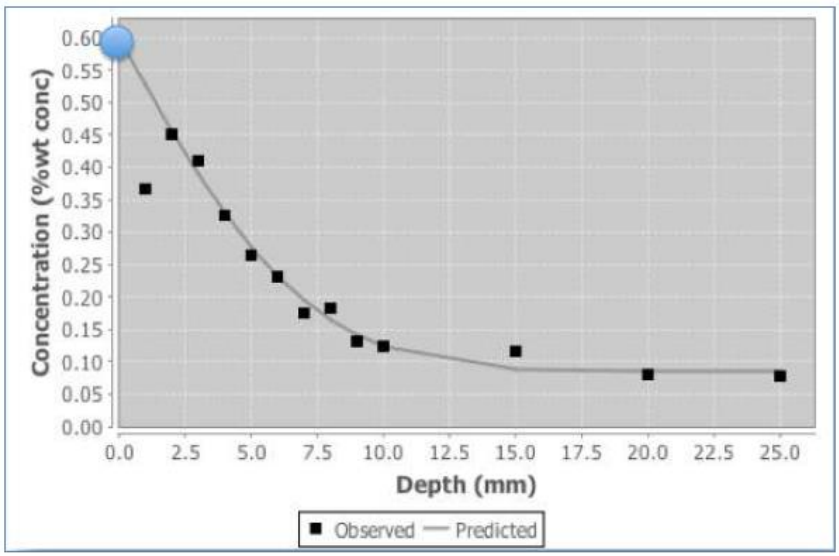
Figure 17 shows examples of surface concentration and temperature profiles for a geographical location in Key West, Florida.



Source: Life 365 Software

Figure 17. Examples of chloride surface concentration and temperature profiles.

Figure 18 shows the predicted chloride concentration at depth zero using the Fick's second law-based for the plot.



Source: Life-365 Manual

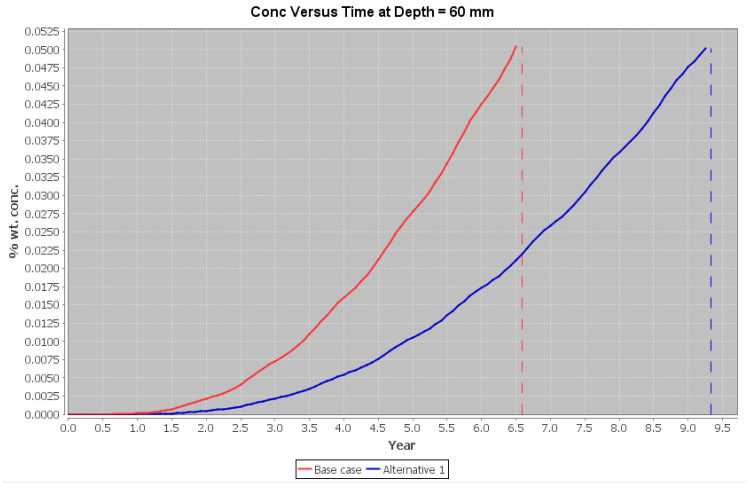
Figure 18. ASTM estimate of surface chloride concentration.

Step 2: Predicting the propagation period or time for corrosion to reach an unacceptable level of chloride concentration.

The typical propagation period for corrosion to reach and acceptable level of chloride concentration is 6 years. The propagation period can change to 20 years when epoxy coated steel is used in the reinforced concrete.

Step 3: Determining the repair schedule.

The time for concrete repair is estimated based on the summation of the initiation and propagation chloride diffusion periods. Figure 19 shows that initiation of the corrosion period is 6.6 years for the base case and 9.94 for alternative 1. Since the propagation time is set to 6 years in both, then repairs should be schedule after 12.6 years for the base case and 15.94 years for alternative 1.



Source: BridgeLLC Software

Figure 19. Chloride diffusion estimates to schedule concrete repairs.

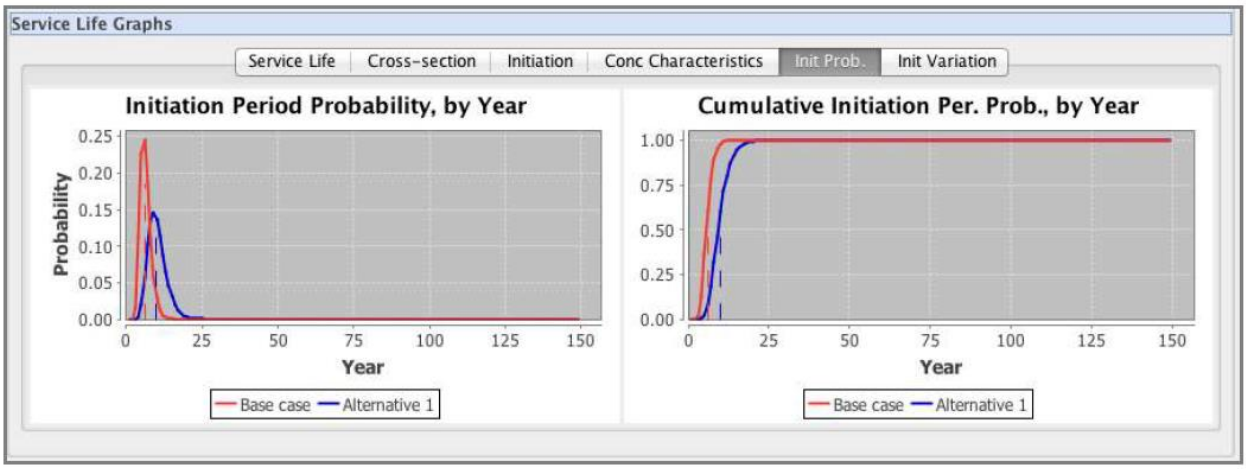
Step 4: Estimating life-cycle cost based on the initial concrete (and other protection) costs and future repair costs.

This step involves the calculation of life cycle costs, and it is described in detail in Chapter 4.

Methodology of Life 365

Average values of the diffusion coefficient (D28), surface chloride concentration (Cs), clear cover to reinforcement (x), critical threshold value Ct) are used to predict the initiation period. These values are then adjusted using a 10 percent range. From the average and standard deviation, time log normal probability and cumulative-probability density functions are generated to estimate the corrosion initiation time.

Figure 20 shows examples of cumulative probability density curves for two alternatives. The highest point in the probabilistic curve determines the most likely time to initiate corrosion. From the cumulative distributions shown in the example, alternative 1 has a higher probability of reaching a service life longer than 16 years than the base case.



Source: Life-365 User Manual

Figure 20. Corrosion initiation probability curves.

Boundaries and Recommendations

The life 365 model predicts the initiation of chloride corrosion with the following assumptions:

- The material under consideration is homogeneous (e.g., no surface effects)
- The surface concentration of chlorides around the concrete member is constant over time.

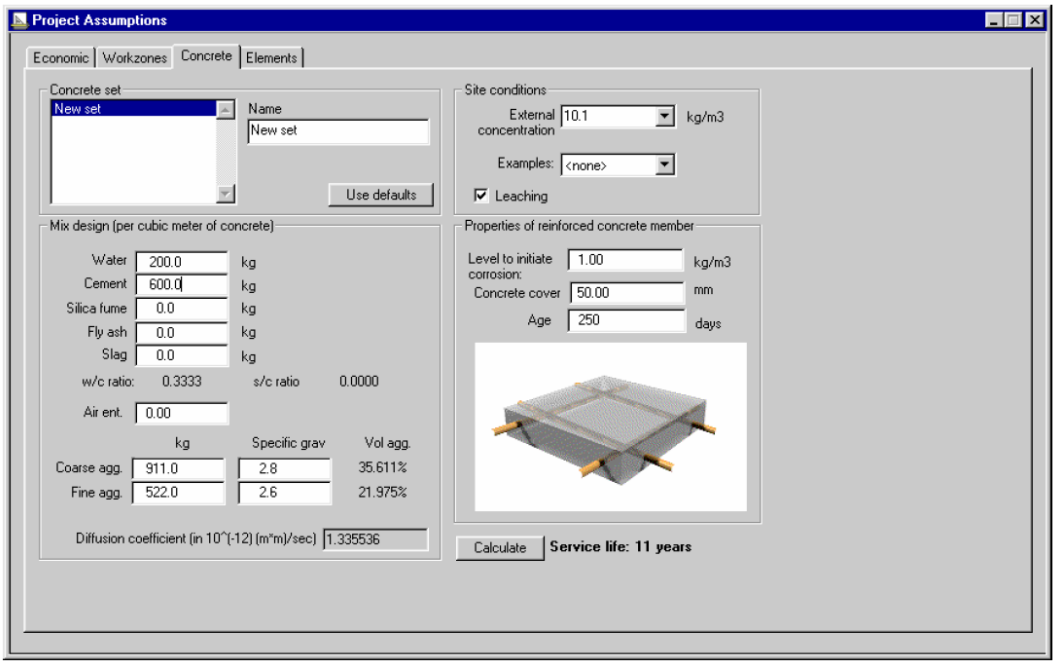
- The properties of the elements are constant during each time step. The diffusion constant is uniform over the depth of the element.

Patch repairs, overlay, shell encapsulation repair life expectancy analysis should consider bonding conditions with existing concrete. The Life 365 model provides a good estimate of the life expectancy for conventional concrete mix designs. However, UHPC is becoming more common in retrofitting bridge applications, a d it should be added as a mix design option in Life 365.

3.2.2 Bridge Life Cycle Cost Analysis (BridgeLCC) Software Tool

Bridge LCC software tool estimates the life cycle costs of bridge structure alternatives. The user defines the project characteristics and alternatives for the comparison, and then calculates the construction and maintenance costs. Costs include agency costs, user costs, and possible indirect costs incurred by third parties impacted by construction and repair activities.

BLCCA has a concrete service life prediction model based on concrete material properties and chloride diffusion. The concrete mix design properties (e.g., w/cm, silica fume, fly ash, slag) are data inputs for the model. External site conditions are considered for different chloride exposure (Kg/m^3) scenarios. Critical chloride concentration and concrete mix information are required to estimate the service life as shown in Figure 21. Results from the analysis allow estimate the time to repair a bridge concrete element.



Source: BridgeLCC software

Figure 21. Service life prediction, Bridge LCC software.

3.2.3 FHWA Long-Term Bridge Performance Program

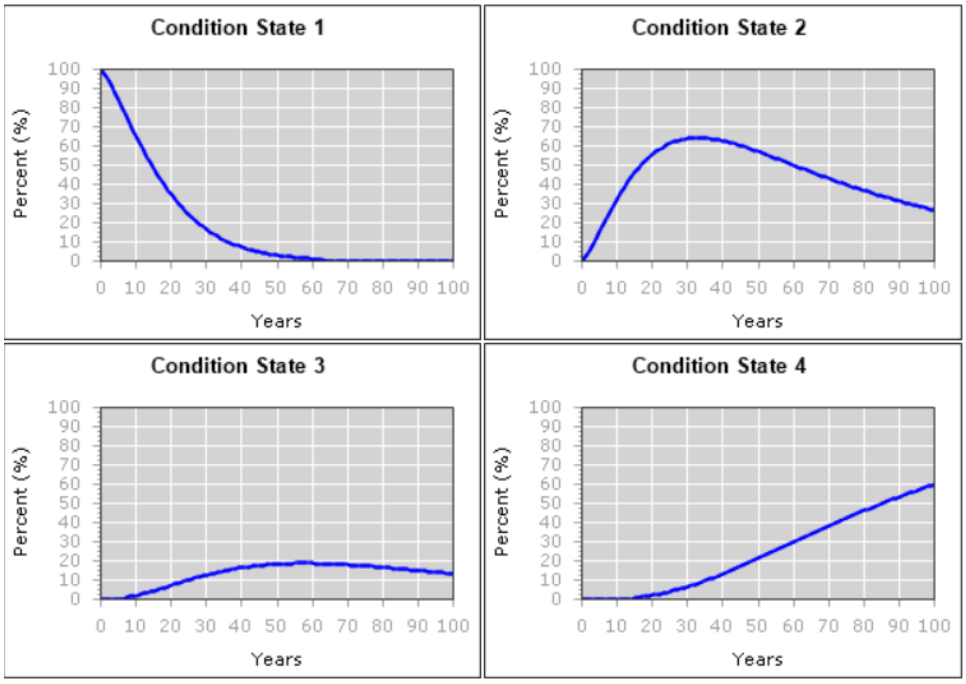
The Federal Highway Administration (FHWA) Long-Term Bridge Performance (LTBP™) Program Info Bridge™ provides a web portal to access, visualize, and analyze bridge performance data. This bridge performance database contains vast information and variables to define life expectancy of bridges. Forecasting of the life expectancy based on bridge element-based rating is also available with filter options to select and sort the data. Some of the selection criteria are State name, Structure Number, Features Intersected/Facility Carried, Bridge Condition, Bridge Age, Main Span Materials or Design, and Scour Critical Bridges. These parameters are part of the National Bridge Inventory. There are also performance data from field tests on concrete structures (e.g., Non-Destructive evaluation, Design or Construction Data), and special projects. It includes UHPC information for 157 bridges. The main important features or attributes of the LTBP database are shown in Table 19.

Table 19. Attributes of data obtained from the LTBP web portal.

Data Category	Attributes
Identification and Location	State name, structure Number, Features Intersected
Structure type and materials	Main Design material type value, Number of spans, deck structure type code, deck protection code
Dimensions and Clearances	Skew Angle, Structure length, Deck Area
Condition Rating and Evaluation	superstructure condition, substructure condition
Climate	Number of freeze-thaw cycles, number of snowfall days
Traffic and Roadway data	Average Daily traffic

3.2.4 AASHTOWare Bridge Management

The AASHTOWare Bridge Management software developed by AASHTO. This software has been used widely as a bridge management tool across the United States and internationally. It is an all-integrated software which has multiple tools for transportation engineers to store bridge inventory and inspection data and formulate network-wide preservation and rehabilitation plans. The deterioration model associated with AASHTOWare is based on NBI condition ratings (AASHTO BrM User Manual 5.2.1). If a bridge is selected for analysis, the software retrieves the deterioration curves for the elements of the bridge. An agency can also define their own elements and set their deterioration rates. The deterioration model is based on a Markovian approach. By default, the software provides the FHWA profile for deterioration and this profile cannot be modified. AASHTOWare has also a module for calculating life cycle costs. The user can add maintenance and rehabilitation activities for LCCA. Figure 22 shows typical deterioration curves used by AASHTOWare. For a bridge element the software gives a median timeframe for each condition state. For example, the median years for CS1, CS2, CS3 and CS4 are 14.42, 42, 14.86 and 1.3. These values can be modified by the user.



Source: AASHTO BrM User Manual 5.2.1

Figure 22. Deterioration curves from AASHTOWare Bridge Management software.

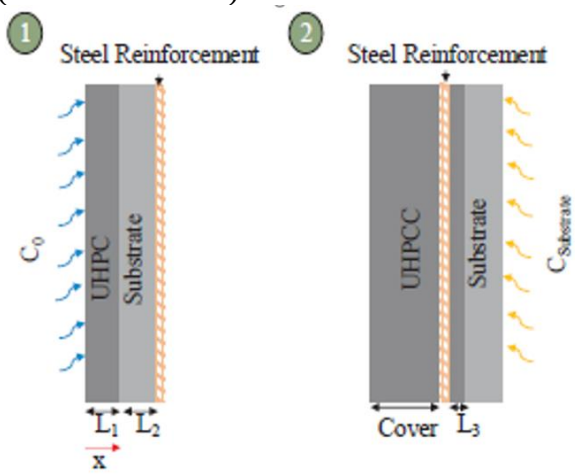
3.2.5 Project Level Analysis Tool (PLAT)

Project Level Analysis Tool (PLAT) was developed by FDOT (Sobanjo 2016) and it is compatible with AASHTOWare bridge management software. This tool is focused more LCCA to support decision-making at the project level. The deterioration model is based on Markovan transition probabilities. Analysis of historical bridge inspection data in Florida revealed that markovian models could overstate the rate of deterioration for bridges in excellent condition.

3.3 Efforts to Model Life Expectancy of UHPC

Bridge components are subjected to traffic, weather, floods, earthquakes, and collisions among other factors during their service life and must be replaced or retrofitted when they are severely damaged. Certain bridge elements are replaced or retrofitted at regular intervals to extend their lifespan and protect larger and more expensive components. Expansion joints, coating systems, deck wearing surfaces, cathodic protection systems, bearings, drainage systems, pile jackets, fenders, and slope protectors are examples of protective elements.

At present, there is no integrated model to predict life expectancy for UHPC applications. The effect of the UHPC shell thickness on structural performance and durability of the retrofitted structure can be calculated based on the time required for chloride ions to reach the surface of the steel reinforcement (Farzad et al 2020). Figure 23 shows two cases for UHPC shell repair. The first case was retrofitting a column with a jacket or layer of UHPC, and the second case was replacing the entire column with UHPC (Farzad et al 2020).



Source: Farzad et al 2020

Figure 23. UHPC shell retrofit cases for bridge column.

Equations were derived from the Fick's law of diffusion and modified using Laplace transformations. The chloride diffusion coefficient for UHPC was set to 1.3×10^{-13} m^2/s in the following equations (Farzad et al 2020):

$$C_1(x,t) = C_0 \sum_{n=0}^{+\infty} \alpha^n \left[\mathrm{erfc}\left(\frac{(2n+1)L_1 + x}{2\sqrt{D_1 t}}\right) - \alpha \mathrm{erfc}\left(\frac{(2k+1)L_1 - x}{2\sqrt{D_1 t}}\right)\right]$$

$$C_2(x,t) = \frac{2kC_0}{k+1} \sum_{n=0}^{+\infty} \alpha^n \mathrm{erfc}\left(\frac{(2n+1)L_1 + kx}{2\sqrt{D_1 t}}\right)$$

The thickness with the longest service life is found by solving these equations for different chloride concentrations and thickness of repair. Deterministic calculations of the time to initiate corrosion are also possible (Farzad et al. 2020). Table 20 shows the values of $t_{critical}$ calculated for surface chloride concentrations 10, 20 and 30 kg/m^3.

Table 20. Life expectancy of UHPC.

Chloride Concentration (C_0) (kg/m^3)	Material of Protective Layer	Thickness of Protective Layer (cm)	$t_{critical}$ (years)
10	UHPC	2	31
		5	140
	NSC	2	6
		5	19
20	UHPC	2	21
		5	95
	NSC	2	4
		5	13
30	UHPC	2	17
		5	80
	NSC	2	3
		5	11

Source: Farzad et al. 2020

The chloride content is converted to content of binder weight (cement + mineral admixtures) in 1 m^3 for UHPC and Conventional Strength Concrete (CSC) as shown in Table 21 (Dobias et al 2016). The distance from the surface should not exceed the thickness of the cover concrete layer to prevent degradation.

Table 21. Chloride content of UHPC compared to CSC at depth of concrete cover.

Depth from the concrete surface (mm)	Chloride content (wt. %)		Chloride content on the weight of binder in 1m^3 of concrete (%)	
	UHPC	CSC	UHPC	CSC
0	0.008	0.01	0.02	0.046
1	0.42	0.41	1.036	1.901
2	0.25	0.33	0.616	1.53
3	0.1	0.29	0.246	1.345
4	0.023	0.26	0.057	1.205
5	0.012	0.23	0.03	1.066
6	0.008	0.21	0.02	0.974
8	0.008	0.18	0.02	0.835
10	0.008	0.175	0.02	0.811
12	0.008	0.16	0.02	0.742

Source: Dobias et al 2016

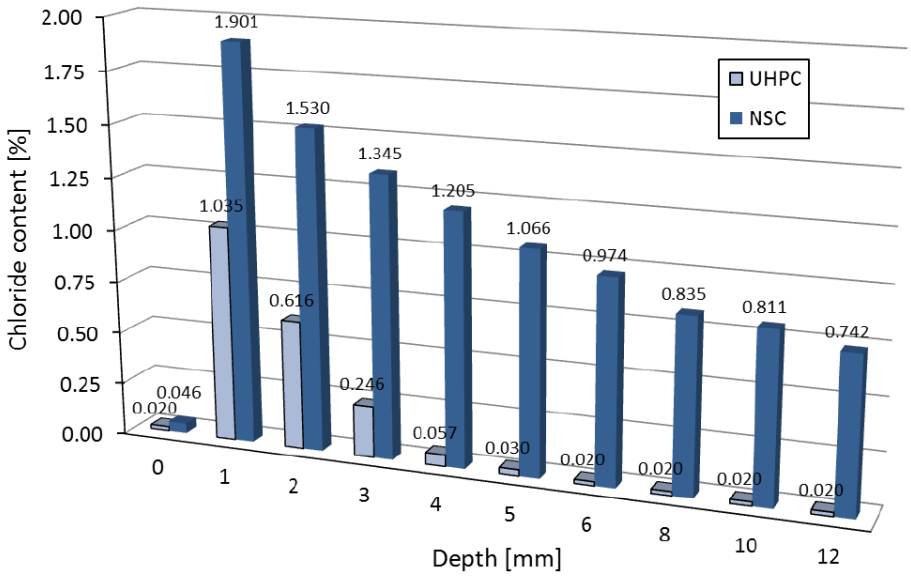
Differences on mechanical properties explain the behavior of concrete in bridge applications. Table 22 shows expected mechanical properties of UHPC and CSC.

Table 22. Durability and mechanical main properties of CSC and UHPC.

Property	Expected Range of CSC	Expected Range of UHPC
Density	140–150 lb/ft3	144–157 lb/ft3
14-day compressive strength	3–6 ksi	18–22 ksi
Modulus of Elasticity	2000-6000 ksi	4,250–8,000 ksi
Poisson's Ratio	0.20–0.21	0.12–0.2
Direct tension cracking strength	0.3–7 ksi	0.8–1.2 ksi
Chloride ion penetrability	less than 4000	50–500 coulombs

Source: (FHWA 2014)

Figure 24 shows that the highest drop of chlorides content in UHPC samples occurs in the first three millimeters of the examined surface (Dobias et al. 2016). For the chloride content in CSC, the drop rate is moderate, and the resistance to penetration of chloride salts is lower than UHPC. The chloride profile provides insights about the propagation period beyond the cover depth. Propagation time is controlled by the chloride diffusion rate after initiation. Moisture absorption and apparent moisture diffusivity of UHPC compared to CSC is about five-times lower (Dobias et al 2016). Therefore, the propagation time is longer for UHPC than for CSC.



Source: Dobias et al 2016

Figure 24. Chloride profile comparing of CSC and UHPC.

3.4 Monte Carlo Based Life Expectancy Model for UHPC

Considering the limitations of existing software to address UHPC applications. A Monte Carlo simulation-based model is an option to estimate life expectancy. Fick's 2nd law determines the time to initiate corrosion. Corrosion is expected to begin at the rebar surface when the chloride content reaches a threshold level. The concrete cover works as a physical barrier to prevent direct exposure of the reinforcement to the surrounding environment, as well as the detrimental impacts of deicing salt, seawater, and other environmental factors. By solving Fick's second law in an inverse manner, the time to initiation can be determined from the following equation:

$$t_1 = \frac{c \cdot erf^{-1}\left(1 - \frac{C_{th}}{C_0}\right)^{-2}}{4D}$$

Where:

t_1 = Time to corrosion initiation
Co = Surface Chloride Concentration
D = Diffusion Coefficient

It should be emphasized that applying Fick's second rule to concrete assumes that the diffusion coefficient and surface chloride concentration remain constant throughout time. Other assumptions made in this model:

- The steel is initially protected from corrosion by the chloride-free concrete that surrounds it. Corrosion occurs when the concrete in contact with the steel is infiltrated with chloride ions to a threshold concentration CT (given as mass of chloride per unit volume of concrete).

- Simple diffusion drives chloride contamination inward, with an apparent diffusion coefficient D, driven by the gradient of chloride ion concentration in the concrete. D is a characteristic of the concrete between the surface and the steel, and its value is constant throughout time and space.
- The crack percentage on the concrete surface is used to estimate post-cracking behavior. Nonetheless, the majority of mathematical and empirical functions shows a linear relationship between rebar loss section and crack width propagation.

3.4.1 Statistical Data Input for Monte Carlo Simulation

Chloride concentration at the concrete surface, concrete compressive strength, concrete diffusion coefficient, chloride concentration threshold at the steel level, corrosion rate, concrete cover depth, are simulated assuming probability distributions. The cover depth can be simulated with a normal distribution. The surface chloride concentration, diffusion coefficient and threshold chloride concentration can be simulated with lognormal distribution (Morcous and Lounis 2007). Tables 23 and 24 displays Monte Carlo data inputs for conventional strength concrete and UHPC.

Table 23. Data inputs for Monte Carlo simulation CSC.

Variable	Unit	Mean value	Standard Deviation	Distribution Type	Reference
Surface Chloride Concentration	kg/m3	5.47	1.83	Lognormal	(Lounis 2003) (Farzad 2018)
Diffusion Coefficient	mm2/year	31.536	9.2	normal	Farzad 2018
Water Cement Ratio	n/a	0.5	n/a	Fixed value	FHWA 2019
Concrete Cover	mm	51 mm (2 inch)	2	Normal	
Threshold Value	kg/m3	1.35	0.135	lognormal	(Lounis 2003)
Propagation Time	Years	6	4	Normal	Used in Life 365 software

Table 24. Data inputs for Monte Carlo simulation, UHPC.

Variable	Unit	Mean value	Standard Deviation	Distribution Type	Reference
Surface Chloride Concentration	kg/m3	5.47	1.83	Lognormal	(Lounis 2003) (Farzad 2018)
Diffusion Coefficient	mm2/year	3.15	0.5	Normal	Farzad 2018
Water Cement Ratio	n/a	0.25	n/a	Fixed value	FHWA 2019
Concrete Cover	mm	51 mm (2 inch)	2	Normal	
Threshold Value	kg/m3	1.35	0.135	lognormal	(Lounis 2003)
Propagation Time	Years	6	4	Normal	Used in Life 365 software

Surface chloride concentrations are expected to increase over time. Chloride concentration rises quickly and peaks at a given depth of concrete cover from the surface (Phares et al. 2006). The diffusion coefficient of UHPC is from a FHWA study (1.3 × 10−13 m2 /s or 3.15 mm2/year) (FHWA 2019). The material properties of UHPC that correspond to this diffusion coefficient are the following:

- UHPC cementitious matrices with cementitious materials contents greater than 1,500 lb/yd3 (890 kg/m3)
- No aggregates larger than a fine sand with an average diameter of 0.02 inches (0.5 mm)
- water-to-cementitious materials ratios less than 0.25.

Some variable values are debated to whether they should be considered constant or time dependent.

The results of the simulation are cumulative probability plots showing the corrosion initiation time for bridge elements as shown in Figures 25 and 26 for conventional concrete and UHPC respectively. It is observed that the initiation period for UHPC is substantially higher than conventional concrete (Farzad 2018). For conventional concrete, the probability of corrosion is about 90% after 20 years. For UHPC, the probability of corrosion initiation is about 90% after 200 years. This is an overestimation of life expectancy for UHPC.

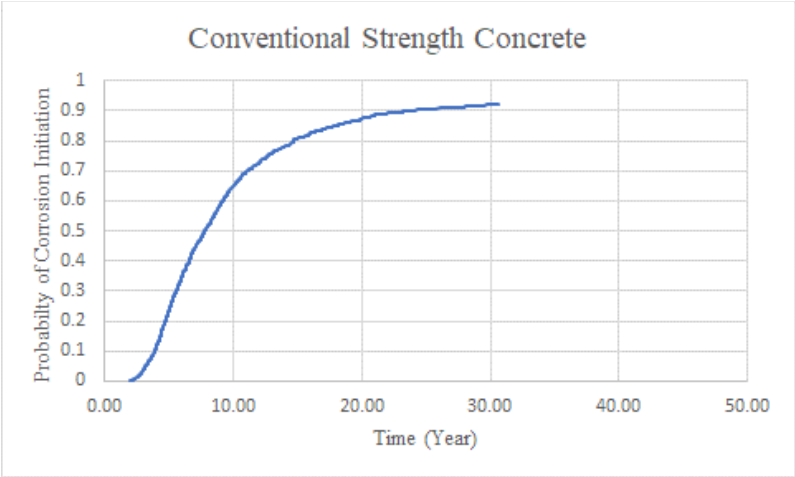


Figure 25. Corrosion initiation time for Conventional Strength Concrete (CSC).

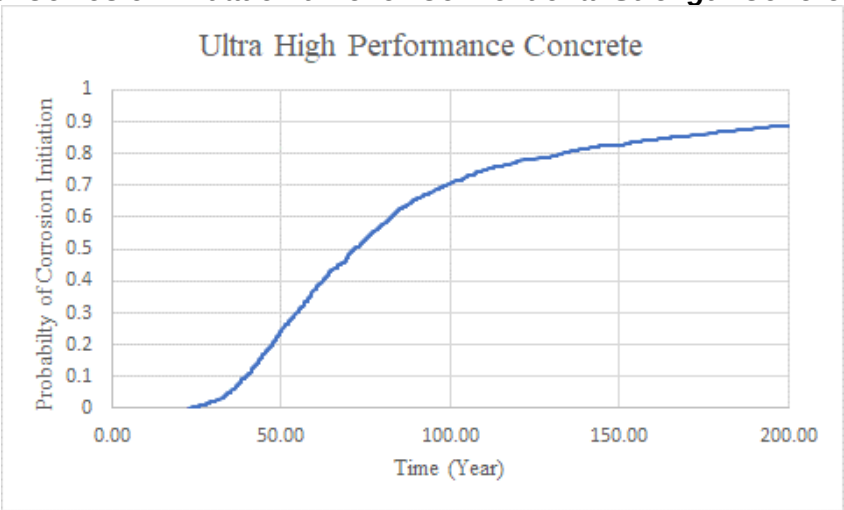


Figure 26. Corrosion initiation time for Ultra High-Performance Concrete (UHPC).

Therefore, for further analysis the bridge element is divided into a certain number of small elements with similar properties. But the small sections have separate probabilities for corrosion due to Monte Carlo simulation. Since the small elements are independent, the cumulative damage is just the multiplication of the probabilities of the small sections. The process is being simulated on python. Crack diffusivity on the concrete element is modeled with derating factors that assume a crack percentage in the concrete when built. The process is further described in FDOT report titled "Corrosion Forecasting for 75-Year Durability Design of Reinforced Concrete" (Sagues et al. 2001) and article "Modeling the effects of Corrosion on the Lifetime of Extended Reinforced Concrete Structures" (Sagues 2003). The research monitored chlorides in regions close to the crack or at the reinforcing surface since local transport conditions influence damage evolution. The simulation process was run in python 3. From the simulation results. figure 27 shows that more than 40 % of spalling damage is expected in 30 years, while UHPC is expected to have 40% of spalling damage in 80 years as shown in Figure 28.

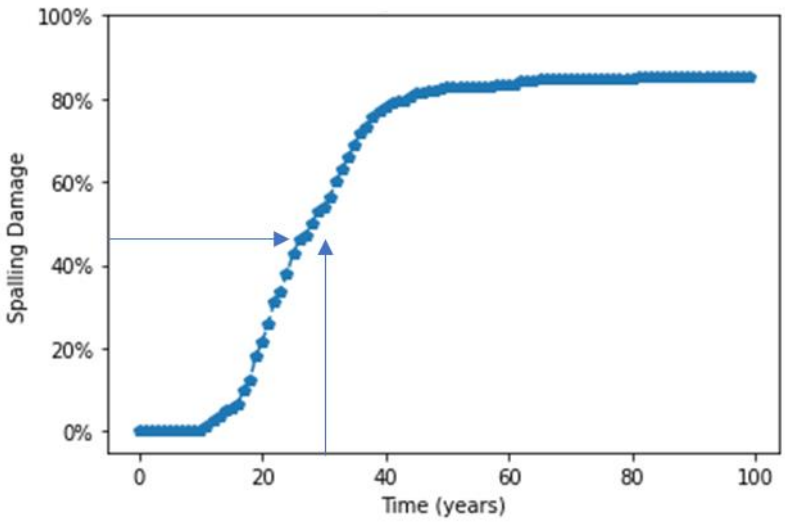


Figure 27. Fraction of elements with spall damage for CSC bridge element.

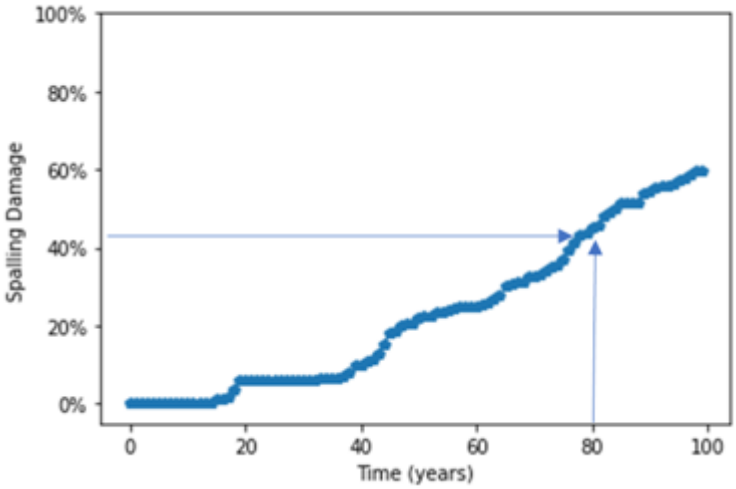


Figure 28. Fraction of elements with spall damage for UHPC bridge element.

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CHAPTER 4: LIFE CYCLE COST ANALYSIS METHODOLOGY TO COMPARE BRIDGE RETROFITTING ALTERNATIVES

A Life Cycle Cost Analysis (LCCA) methodology to evaluate conventional concrete with UHPC applications for ABC projects is described in this chapter. Retrofitting techniques are categorized into preventive maintenance and rehabilitation activities. FHWA guidelines are followed to quantify the life cycle costs including agency and user costs. The use of software tools to facilitate the LCCA calculations is also addressed in this Chapter.

4.1 Overview of Life Cycle Cost Analysis

Life Cycle Cost Analysis (LCCA) is an economical assessment of a project that considers all present and future costs required to provide the desired level of service. The purpose of LCCA is to identify the project alternative that meets the technical requirements at the lowest cost over the expected period of service. Initial construction costs, future maintenance costs, rehabilitation costs, and user costs over the life cycle of a project are considered in the analysis. The main parameters to establish to perform LCCA are:

- Length of the analysis period
- Costs to be included in the analysis
- Salvage value
- Discount and inflation rates

Length of Analysis Period

The length of the analysis period must be carefully selected, and it is related to the project service life. An analysis period of twenty to sixty years is recommended at the network management level. Longer periods of time increase the uncertainty of forecasting traffic, bridge performance, treatment costs, and other factors involved in the analysis.

Costs to be Included in the Analysis

All costs anticipated over the life of the bridge including initial costs, maintenance and rehabilitation costs, salvage value, and user costs should be considered in the LCCA. The costs used in the analysis should be current and accurate. Agency construction and maintenance costs are gathered with reasonable precision by transportation agencies, while user costs are more complex to determine.

Salvage Value

Salvage value represents the worth of the bridge at the end of the analysis period. The salvage value decreases as the bridge condition deteriorates. The salvage value id the value of the salvageable materials at the end of the analysis period, or the value of the bridge remaining service life. Agencies estimate the remaining life using performance models to forecast the bridge condition. An approach to obtain the salvage value is to subtract the funds needed to restore the bridge condition to current standards from the replacement cost. Therefore, the salvage value can be estimated at any time of the bridge service life.

Discount and Inflation Rates

Discount rates are used to bring back costs incurred at different times of the analysis period to a base year. Costs and benefits are normally expressed in constant monetary terms, usually as today's dollars at the base year of analysis. Discount rates are used to calculate the present value of future life cycle costs. The effect of inflation must also be removed to compare alternatives on the same basis over time. In the LCCA, future costs and benefits are both affected by the same inflation; therefore, the effect of inflation could be ignored in the analysis if the purpose is to identify the best alternative.

A real discount rate can be estimated by removing the rate of inflation (as measured by a general price index such as the CPI) from a market (or nominal) interest rate for government borrowing. The selected market rate for government borrowing should be based on government bonds with maturities comparable in length to the analysis period used for the economic analysis. Real discount rates calculated in this manner have historically ranged from just below 0 percent to 5 percent (NCHRP 2003). These are the rates most often used by states for discounting highway investments. The U.S office of Management and Budget (OMB) currently requires U.S Federal agencies to use a 7 percent real discount rate to evaluate public investments and regulations.

The discount rate depends on the source of financing. If the project is funded by a local government, the municipal bond rate may be more applicable as a discount rate. If private investors undertake the project, corporate bond is a good indicator. If federal funds are associated to the project, the market rate for government borrowing may be used for LCCA (Ozbay et al. 2003).

LCCA Economic Evaluation

Discounted cash-flow methods including net present value, equivalent uniform annual cost, and internal rate of return are often used in the economic evaluation of the alternatives under consideration. A brief description of these methods follows.

Net Present Value (NPV)

The Net Present Value (NPV) is the sum in constant monetary terms of all present and future costs incurred in the analysis period. To calculate the NPV, future costs are converted to the present at the selected discount rate. The present cost of a future cost is obtained with the following formula:

$$\$P = \$F\,(1+r)^{-n}$$

Where:
$P = Present Cost
$F = Future Cost
r = Discount Rate
n = Number of periods between the present and future cost.

NPVs of the alternatives under consideration are compared over the period of analysis. If the alternatives do not have equal lives, the comparison assumes identical replacement when an alternative reaches the end of its life, and the life cycle restarts again. This problem is also solved by estimating the salvage values at the end of the analysis period.

Equivalent Uniform Annual Cost (EUAC)

Present and future costs are converted to an Equivalent Uniform Annual Cost (EUAC) using a discount rate. This method allows flexibility in the economic analysis, particularly when the alternatives have different service lives. If decisions are made based on annual budgets, EUAC facilitates the comparison of LCCA results, and the alternative with the lowest EUAC is recommended.

Internal Rate of Return (IRR)

The Internal Rate of Return (IRR) is the interest rate at which the benefits are equivalent to the costs. IRR does not indicate the amount of the costs or monetary benefits of a project but represents a profitability indicator about the investment. IRR is used when an agency looks for project financing sources with variable borrowing costs. If the IRR is larger than the discount rate established for the expected profit, then the project is economically justified. The higher the IRR, the more profitable the project is considered by the agency.

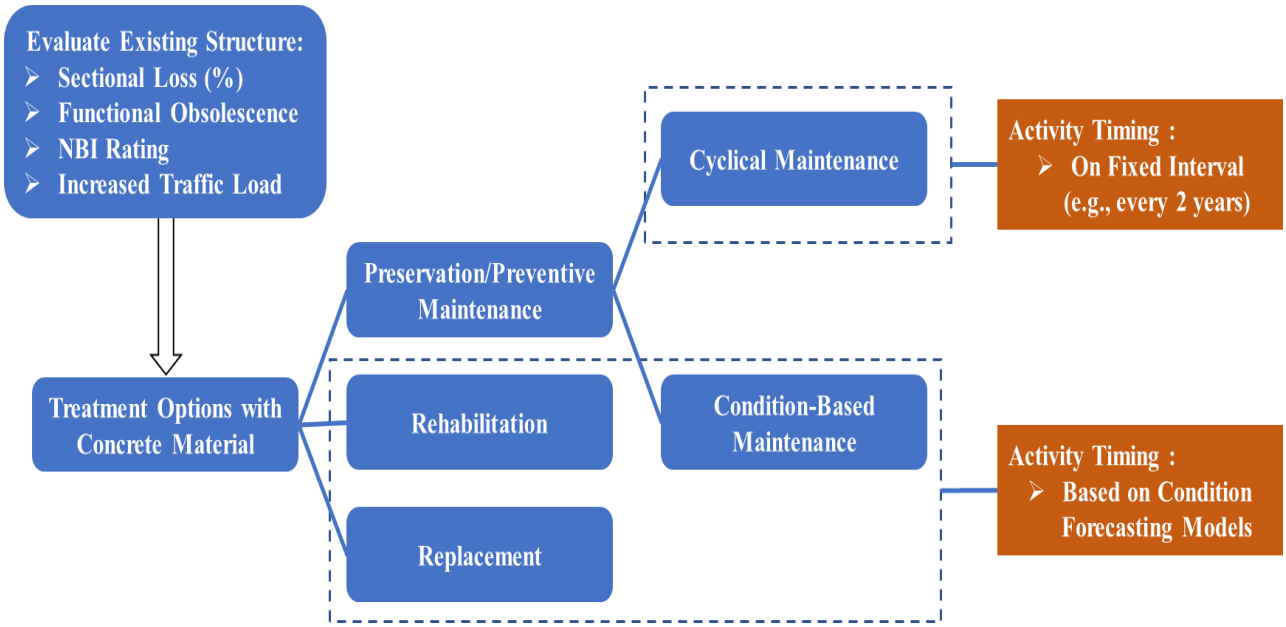
4.2 LCCA Methodology to Compare Bridge Retrofitting Alternatives

An overview of LCCA step-by-step methodology to compare bridge retrofitting alternatives is shown in Figure 29. First, retrofitting alternatives must be defined for the economic comparison. The workflow of each alternative should be documented with their corresponding associated costs. The analysis period is then established based on the type of project and bridge element to be repair. The deterioration model to forecast the expected performance should be selected accordingly. Maintenance and rehabilitation activities and cost estimates are determined for the entire period of analysis. The NPV and EUAC for each retrofitting alternative is calculated and compared. Finally, a retrofitting technique is recommended based on the LCCA results.

Figure 29. Overview of the LCCA step-by-step methodology to compare bridge retrofitting alternatives.

4.2.1 Retrofitting Techniques with UHPC for ABC Projects

Bridge retrofitting techniques are determined according to the bridge element condition, type and extend of the damage. UHPC can be utilized in overlays, claddings, and shells to preserve or rehabilitate bridge concrete decks, girders, or columns (Graybeal B. 2011). Entire deck, beam and bridge systems can be built with UHPC to replace damaged elements. As shown in Figure 30, FHWA classified the bridge actions into three categories: preservation/preventive maintenance, rehabilitation, and replacement (FHWA 2018).



Source: Adapted from FHWA 2018

Figure 30. Bridge action categories.

A brief description of these categories follows:

Bridge Preservation: "Actions or strategies that prevent, delay or reduce deterioration of bridges or bridge elements; restore the function of existing bridges; keep bridges in good condition; and extend their life" (FHWA 2018). Bridge preservation includes cyclical preventive maintenance and condition-based maintenance activities.

Cyclical preventive maintenance: Activities "performed on a pre-determined interval and aimed to preserve existing bridge element or component conditions. Bridge element or component conditions are not always directly improved as a result of these activities, but deterioration is expected to be delayed." (FHWA 2018). Condition based maintenance activities are based on ratings (e.g., NBI rating, Structural deficiency rating).

Rehabilitation: "Rehabilitation involves major work required to restore the structural integrity of a bridge as well as work necessary to correct major safety defects" (FHWA 2018). If rehabilitation work is not performed timely, a bridge becomes Structurally Deficient.

Replacement: Involves replacement of damaged bridge elements with updated structural design and increased load capacity and higher durability (FHWA 2018).

Preventive Maintenance and Rehabilitation with UHPC

UHPC can be used in both preventive maintenance and rehabilitation activities. The most common activities are deck closure pours for precast deck elements, bridge deck overlay, and shell encapsulation. Table 25 summarizes the activities, and brief descriptions for each follow.

Table 25. Preventive maintenance and rehabilitation techniques with UHPC for ABC projects.

Bridge Activity	UHPC Technique	Conventional Alternatives
Column/Pier *(Rehabilitation)*	UHPC Shell Encapsulation	Normal Concrete Shell
		steel Casing
Deck Closure Joints *(Preventive maintenance and Rehabilitation)*	Ultra-High-Performance Concrete (UHPC) with straight bars.	Normal Strength Concrete (NSC) with straight bars
		Normal Strength Concrete (NSC) with headed bars.
		Normal-Strength concrete with 180-degree hooked bar
		Epoxy Joint
Deck Overlay *(Preventive maintenance and Rehabilitation)*	UHPC Deck Overlay	Standard Concrete Overlay
		High Performance Concrete (HPC) Overlay
		Asphalt Overlay w/membrane
		Asphalt Overlay w/o membrane
		Latex-Modified Overlay
		Micro Silica Overlay
		Polymer Concrete Overlay
		Thin Bonded Epoxy Overlay
Link slab connection *(Rehabilitation)*	UHPC link slab connection	Link slab using compressive strength >4 ksi, Low Shrinkage Class A4 Concrete

UHPC Shell Encapsulation for Bridge Columns

Field-cast UHPC is a rehabilitation technique applied for improving the strength and ductility of superstructure supporting elements such as driven piles and bridge columns. The seismic performance of bridge columns with deficient lap splices in the plastic hinge zone can be improved by UHPC jacketing (Dagenais et. al. 2014). For combination of axially and laterally loaded sub-structural reinforced concrete (e.g., bridge columns), replacement of existing surface concrete and shell encapsulation with UHPC is an alternative. UHPC should decrease the steel corrosion deterioration rate by confining the concrete and providing a barrier with low permeability (Farzad et al. 2020).

The repair scheme using the UHPC is rather efficient regarding lateral strength, deformation, energy dissipation capacity and stiffness degradation. Numerical models using MATLAB can be used to analyze the force-displacement response of concrete members. The properties of UHPC make it an innovative repair material for retrofitting the damaged body of marine piles (Farzad 2018).

UHPC Bridge Deck Closure Pours

UHPC is frequently used as bridge deck closure pours in preventive maintenance and more frequently in accelerated bridge rehabilitation. The rebar formation in the closure joints varies according to the UHPC application (Jaberi Jahromi et al 2020). It can be used effectively as a shear connector in both longitudinal and transverse connecting joints (Russel and Graybeal 2013).

With the use of prefabricated elements, field-cast UHPC can simultaneously resolve several conventional concerns. Because of UHPC's mechanical properties, field-cast connections can be smaller, use less costly connectors, and outperform the connected components by removing the connections as a weak link in the framework. The fresh properties of UHPC allow fill of tight and potential hidden link spaces without honeycombing or unintended voids. The field-cast UHPC connections can withstand the aggressive conditions that have caused field-cast grouts on traditional

concrete mixes that have prematurely degraded in the past (Graybeal 2011).

UHPC Bridge Deck Overlay

Concrete overlays on bridge decks are used to rehabilitate the structure to avoid deterioration due to fatigue cracking. UHPC overlays can be considered as a preventive measure in conjunction with spot repairs of isolated distresses. Concrete overlays are grouped in bonded and unbonded (Shann 2012). UHPC high durability and mechanical properties can reduce the time of traffic closures and extend the service life of the bridge deck. UHPC overlays with minimum 25mm (1in.) thickness could be a more cost-effective alternative when compared to conventional bonded concrete overlays (Khayat et al. 2018). Also, the absence of mechanical consolidation due to the high fluid nature of UHPC materials can reduce construction time for the rehabilitation of bridge decks.

UHPC has low water cement ratio, high binder content, and optimum packing density that eliminates capillary pores by providing a dense matrix (Naaman and Wille 2012). UHPC can also be used as an overlay on bridge decks to cover damaged or exposed bridge parts after replacing degraded reinforcing bars.

UHPC Link Slab Connection

Studies have developed an innovative link slab design utilizing UHPC to eliminate transverse deck joints wherever feasible (Royce 2016). Link slab design assumes that the UHPC section is subject to bending. The link slab also acts as a semi-rigid link between spans transferring compressive, tensile, and shear stresses due to various loads. The ability of UHPC to develop ultimate tensile strains up to 0.007 by developing internal micro cracks allows the link slab to accommodate girder end rotations (Royce 2016). Limiting the tensile strain increase the service life of the link slab by preventing the penetration of moisture and chlorides. The design of the link slab is influenced by variables such as span arrangement, bearing type and arrangement, girder end rotation due to live load, and bridge skew. Several rehabilitation projects have utilized UHPC link slabs to eliminate joints.

UHPC Drill Bits

Steel is a popular material for drill bits, and it is the standard material for UHPC special foundation applications. Drill bits are applied by torque as well as compression powers. Steel may be replaced by UHPC in many fields of special foundation engineering. UHPC is distinguished by its dense composition, which provides excellent longevity. Furthermore, the compressive strength of UHPC is approximately five times greater than the strength of traditional concrete (Plevny 2020).

Bridge Element Replacement with UHPC

UHPC allows innovative bridge element replacement techniques that accelerates the rehabilitation process, extending the bridge service life with minimum road user delays and community disruptions (Aaleti et al. 2013). UHPC has several benefits due to its greatly enhanced physical qualities. Due to UHPC's strength, smaller sections may be designed resulting in lighter structures.

For conventional bridge design “AASHTO LRFD design Specification” is recommended (AASHTO 2017) (AASHTO 2018). Several FHWA guidelines are used for UHPC connections, overlays, and other replacement techniques (FHWA 2018) (FHWA 2019). Table 26 shows a summary of the most common bridge element replacement techniques with UHPC for ABC projects.

Table 26. Bridge element replacement techniques with UHPC for ABC projects.

Bridge Activity	UHPC Options	Conventional Alternatives
Beam & Girder	Box Girder	Conventional Concrete Girder/Beam
	Bulb T girder	Prestressed beam
Deck	I girder	Conventional Concrete Deck
	Full-Depth Waffle Deck panel	

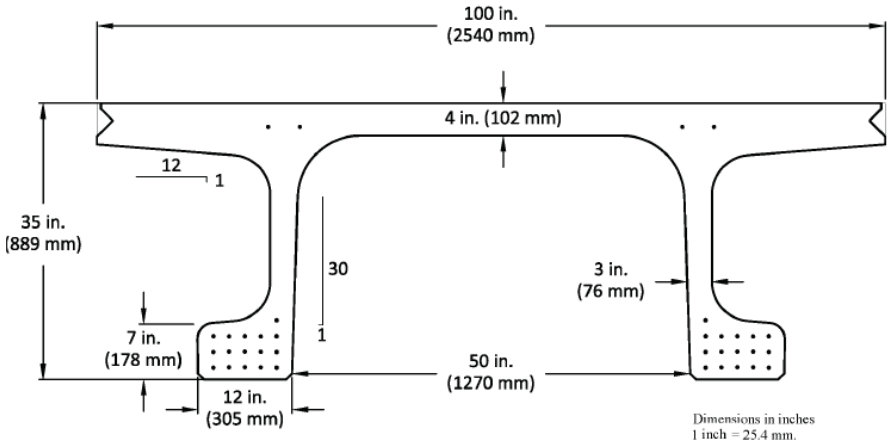
Table 26. Bridge element replacement techniques with UHPC for ABC projects (Continued).

Bridge Activity	UHPC Options	Conventional Alternatives
Pier & Column	Precast pile for deep foundation	Cast in place pile
Systems	Steel truss- UHPC plate bridges	Conventional girder & Deck systems
	Precast UHPC girder with ordinary concrete slab	
	FRP girder with UHPC slab	
Wall & Barrier	Precast Cantilever Retaining Wall	Cast in place retaining walls

A description of the most common UHPC techniques that applied to ABC projects follows.

UHPC pi-girder

UHPC pi girders can be up to 65 ft (Graybeal 2009). This system is good for sites with clearance limitations. This girder is also good for short and medium span bridges. Cost savings can be achieved by using partial prestressing in UHPC pi-girder design. Girders with depths of 47 inches can be used on spans of 135ft or less (FHWA 2014). Figure 31 shows a typical UHPC pi girder section.



Source: FHWA 2014

Figure 31. Typical section of a UHPC pi girder.

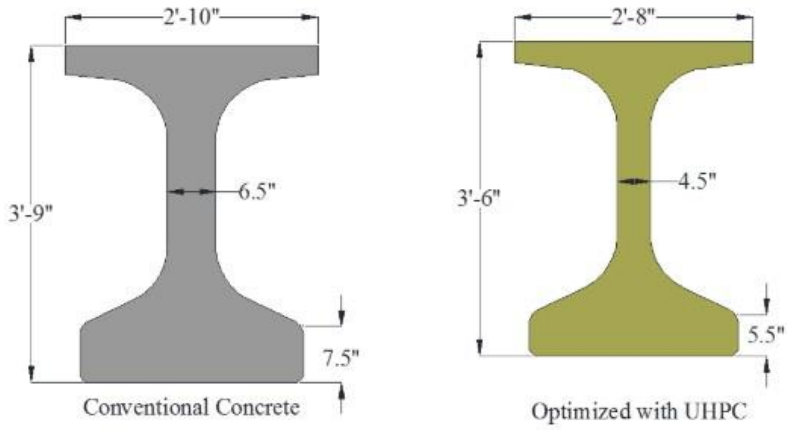
UHPC Waffle Deck Panel System

A UHPC waffle deck system consists of precast UHPC waffle panels with shear pockets, transverse panel-to-panel connections, longitudinal panel-to-girder connections, some type of overlay to improve rideability if desired, and in situ UHPC material to fill the connections and shear pockets (Aaleti et al. 2013). The use of UHPC waffle deck panels has several advantages. UHPC waffle slabs are 30 to 40% lighter than solid precast full-depth panels constructed of standard strength concrete for decks of the same thickness and capacity (Aaleti et al. 2013). When compared to decks made with solid precast panels, the lighter UHPC panels can have longer span lengths, increase girder spacing, and improve bridge load capacity.

UHPC in Prefabricated Element Systems

The high strength of UHPC results in a substantial reduction of dead-load and less restricted structural member shapes (Plevny 2020). As compared to conventional concrete, UHPC allow longer span bridge structures with smaller member sizes, as well as a substantial reduction in volume and self-weight. A UHPC beam, for example, needs half the section depth of reinforced or pre-stressed concrete beams, resulting in a weight reduction of 70% (Ghoneim et al. 2006). UHPC piles can be cast successfully in a pre-casting plant (Voort et al. 2008). High strengths of 26 to 29 ksi (179 to 200 MPa) are achieved by UHPC with heat treatment procedures. To avoid the formation of air pockets in UHPC members, limited vibration of UHPC piles during casting is recommended at locations every five to ten feet along the pile for approximately ten seconds at each site (FHWA 2014).

The use of precast concrete bridge components is a cost-effective way to speed up the replacement of bridges (Plevny 2020). Prefabricated Bridge Element Systems (PBES) advantages are maximized when high-performance materials are used in the prefabrication of bridge elements. High performance concrete and large diameter prestressing strands used in fabricating precast/prestressed I-girders displayed superior shear and flexure capacity, and results in smaller girder sections, higher span-to-depth ratio, and increased girder's centerline spacings (Haber *et al.* 2018). Open to traffic in 2006, the Mars Hill Bridge was the first vehicular bridge in the United States to use UHPC. This 110-foot (33.5 m) span bridge was originally designed using conventional concrete with prestressed girders, and was planned to be a 3- span structure with two intermediate piers. Using UHPC, the bridge was constructed as a single span with three supporting girders at a 9.5-foot (2.7 -meter) spacing. The original girder design, a 45 inch (1143 mm) deep bulb tee bridge girder, was optimized with UHPC. The resulting girder was shallower and 25 percent lighter than the original bulb tee. The original and optimized girders are shown in Figure 32.



Source: Haber *et al.* (2018)

Figure 32. Comparison of prestressed bridge girders for the Mars Hill Bridge composed of conventional concrete and optimized with UHPC.

4.2.2 Analysis Period

The life expectancy of the retrofitting alternatives under consideration is a reference to determine the length of the analysis period. For example, a short period of time may be adequate for determining when a deck overlay should be scheduled for a standard highway design (e.g., ten years) while a longer period is recommended for bridge replacement systems (e.g., 25 to 50 years). AASHTO recommends a 75-year design service life for new bridges (AASHTO 2009).

Life expectancy of UHPC is greater than conventional concrete, and it can reach up to 100 years (Farzad 2020). Based on the life expectancy of new concrete structural materials, the analysis period might be even extended over 100 years (NCHRP 2012). Previous studies have used 75 years to compare bridge systems with UHPC and conventional concrete (Dong 2018). Retrofitting alternatives with service lives longer than the analysis period should estimate the salvage value in the LCCA.

4.2.3 Estimating Agency Costs

Agency costs include initial construction, maintenance, and rehabilitation activities. Salvage value may also be considered in the LCCA depending on the retrofitting techniques under comparison and length of the analysis period. Initial construction, maintenance, and rehabilitation costs can be determined by adding material, labor, and equipment costs. The cost of the materials is affected by the manufacture process. Certain materials need specialized workers or must be manufactured off-site and shipped, material manufacture can have a significant impact on the cost. Use of special equipment may speed up the construction activity but at the expense of additional costs. Construction

procedures also affects the cost of retrofitting activities. The overlay UHPC material is mixed in a large pan-style mixers. Often two mixers are used to provide constant supply of UHPC. The UHPC material is then transported and placed with a motorized buggy. The use of vibratory screed is used after laying down the overlay. A water and wax based concrete curing compound is often applied after laying the overlay. The details of construction vary from project to project. Other agency costs that may be considered in LCCA are related to the design, condition assessment, right-of-way, and utility adjustments. The costs of preliminary engineering (PE), construction engineering (CE), traffic maintenance (MOT), and demolition activities can be added to the costs. Figure 33 summarizes the agency costs that should be considered for LCCA.

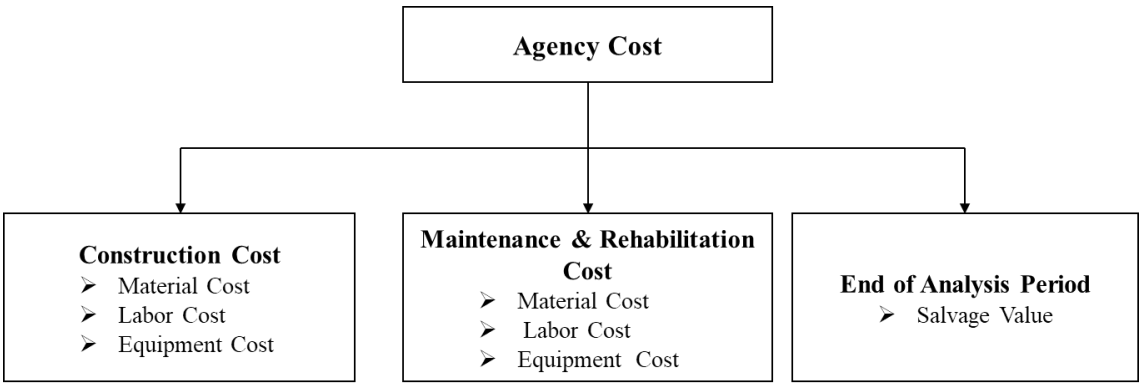


Figure 33. Agency costs.

The costs of bridge preventive maintenance, rehabilitation, and replacement can be estimated on a unit cost basis. Table 27 shows the agency cost units recommended for LCCA.

Table 27. Recommended agency cost units for LCCA.

Agency Costs	Unit
Construction Costs (e.g., construction of repair, Rehabilitation, Replacement, approach road)	$/sq. unit
Bridge demolition cost	$/sq. unit
Bridge annual maintenance cost	$/sq. unit
Maintenance of traffic	Percentage of construction cost
Preliminary Engineering	Percentage of construction cost
Construction Engineering	Percentage of construction cost
Salvage value	$ / sq. unit

Self-propelled modular transporter (SPMT) is a common Accelerated Bridge Construction (ABC) method. SPMT is used to carry pre-cast elements from the casting site to construction zone (Aktan and Attanayake 2017). Therefore, mobilization costs should be considered in the LCCA.

Mobilization cost

"Mobilization cost is calculated based on the number of required SPMT axle lines, transportation distance of the SPMT axles (distance to project site) and unit transportation cost of an axle line" (Aktan and Attanayake 2017). Mobilization cost of SPMT can be estimated from the following equation (Aktan and Attanayake 2017):

Mobilization Cost ($) = k × d × uct ; k =n/6 (nearest round)

Where:

n = Number of required SPMT axle lines

d = Distance from a SPMT equipment hub

uct = Unit cost for transportation.

Travel Path Preparation Cost

Travel path preparation cost can be calculated from the following equation:

Travel Path Cost ($) = L × b × t × ucbp

Where:

L = Length of travel path

b = Width of travel path

t = Minimum required base thickness

ucbp = Unit cost of base preparation

Length of travel path is obtained from site specific data. Minimum required base thickness and other cost data inputs can be calculated by the method described in detail in the MDOT report titled "Research on Evaluation and Standardization of Accelerated Bridge Construction Techniques" (Aktan and Attanayake 2017).

Agency costs for UHPC applications may be retrieved from the ABC-UTC project database. Project bid information contains the construction costs. The costs can be further broken down through analysis of the workflow.

Preventive Maintenance

Bridge preventive maintenance activities can be classified into two sub-categories (AASHTO 2007):

- Cyclical Maintenance
- Condition Based Maintenance

Bridge preventive maintenance activities that are common are briefly described as follows.

Crack Sealing

Cracks are the most common type of damage in concrete bridge elements. Cracks are caused by a combination of loads, temperature variation and other conditions. Cracks that are spaced two or more feet apart should be sealed and crack filler materials are recommended depending on the deck width (Washer et al. 2017). The frequency of crack sealing work varies among Department of Transportations. For example, Minnesota DOT (MnDOT) have sponsored projects that have called for cyclic crack sealing at least once every five years (Alipour et al. 2020).

Deck Patching

Over time, as bridge decks crack and wear, spalling of the deck surface can eventually occur. Spalled concrete decks can be caused by a variety of factors, including repeated wheel damage, freeze-thaw cycles, snow removal, and underlying concrete faults. Deck patching is a type of preservation. Depending on the severity of the damage, patching can be done to various depths of the deck, both partial and full patching. Usually, the deck patching scheme is performed based on the condition state criteria, and the cost depends on the concrete material finally used.

Spot painting

Coatings on new bridges are normally anticipated to last 20 to 30 years (Hopwood et al. 2018) with deviations dependent on environmental conditions and use. On bridges, spot painting is used to

maintain the present topcoat of the steel superstructure against corrosion and degradation. Bridges can be damaged by bare steel corroding fast, especially in regions prone to water exposure, such as below bridge joints. Road salts hasten the process, necessitating more regular bridge repainting. Spot painting may be considered as a cost-effective solution in terms of total cost for maintaining the overall integrity of a structural element (Hopwood et al. 2018).

Cyclical preventive maintenance activities, such as deck cleaning, washing, painting of steel, are carried out on a regular basis regardless of bridge (Chang et al. 2016). Table 28 shows cyclical maintenance activities for bridge components.

Table 28. Example of cyclical maintenance activities.

Bridge Component	Cyclical Maintenance Activity
Deck	Clean/Wash Bridge
	Clean and Flush Drains
	Crack Sealing
	Deck Overlays
Superstructure	Clean/Wash Bridge
Substructure	Seal Concrete
	Clean/Wash bridge

Source: FHWA 2018

Table 29 shows cyclical preventive maintenance activities with commonly used frequencies.

Table 29. Frequency of cyclical preventive maintenance activities of bridge elements.

Cyclical Preventive Maintenance Activity	Commonly Used Frequencies (Years)
Wash/clean bridge deck or entire bridge	1 to 2
Install deck overlay on concrete decks such as:	
Thin bonded polymer system overlays	10 to 15
Asphalt overlays with waterproof membrane	10 to 15
Rigid overlays such as silica fume and latex modified	20 to 25
Seal concrete decks with waterproofing penetrating sealant	3 to 5
Zone coat steel beam/girder ends	10 to 15
Lubricate bearing devices	2 to 4

Source: Chang et al. 2016

Deck overlays are sometimes considered as preventive maintenance. Table 30 shows typical overlay cost ranges for varying thickness.

Table 30. Examples of bridge deck overlays cost range.

Overlay Type	Overlay Thickness Range - in (mm)	Range of Cost -$/sft ($/sqm)
High Performance Concrete	1-5 (25-127)	17-25 (183-269)
Low Slump Concrete	1.5-4 (38 -102)	13-19 (140-204)
Latex Modified Concrete	1-5 (25-127)	18-39 (193-419)
Asphalt with a Membrane	1.5-4 (38-102)	3-8 (32-86)
Polymer-based	0.13-6 (3-152)	10-17 (107-183)
Nonproprietary UHPC	1-2 (25-52)	3-6 (32-64)
Proprietary UHPC		9-18 (97-184)

Source: Haber et al. 2018

Condition based maintenance of concrete surface is mainly due to scaling, cracking, and spalling of the concrete. Generally, 13mm to 25mm scaling is considered for preventive maintenance (AASHTO 2007). Table 31 shows condition-based maintenance activities for bridge components.

Table 31. Examples of bridge condition-based maintenance activities.

Bridge Component	Condition Based Maintenance Activity
Deck	Drains, repair/replace
	Joint Seal Replacement
	Joint Repair/Replace/Elimination
	Deck Overlays
Superstructure	Seal/Patch/Repair Superstructure Concrete
	Protective Coat Concrete/Steel elements
	Spot/Zone/Full Painting Steel Elements
Substructure	Patch/Repair Substructure concrete
	Protective Coat/Concrete
	Pile Preservation (Jackets/wraps)

Source: FHWA 2018

Condition-based maintenance activities are defined using performance measures rating scales:

- NBI General Condition Rating
- NBI structural Condition Rating
- National Bridge Inventory (NBI) Structurally Deficient (SD) or Functionally Obsolete (FO) Status
- Sufficiency Rating
- Bridge Health Index

For example, Table 32 shows NBI condition rating and corresponding recommended actions.

Table 32. NBI general condition rating and feasible actions.

Rating	Description	Commonly Employed Feasible Actions
9	Excellent Condition	Preventive maintenance
8	Very Good Condition- no problems noted	
7	Good Condition - some minor problems	
6	Satisfactory condition	Preventive maintenance
5	Fair condition	
4	Poor condition	Rehabilitation or replacement
3	Serious condition	
2	Critical condition	
1	"Imminent" failure condition	
0	Failed condition	

Source: FHWA 2018

Annual maintenance cost can also be projected based on past records to determine future budget needs. Table 33 shows examples of annual bridge maintenance costs from two DOTs, though the costs are outdated. It should be noted that most retrofitting techniques mainly only require cyclic maintenance after their application.

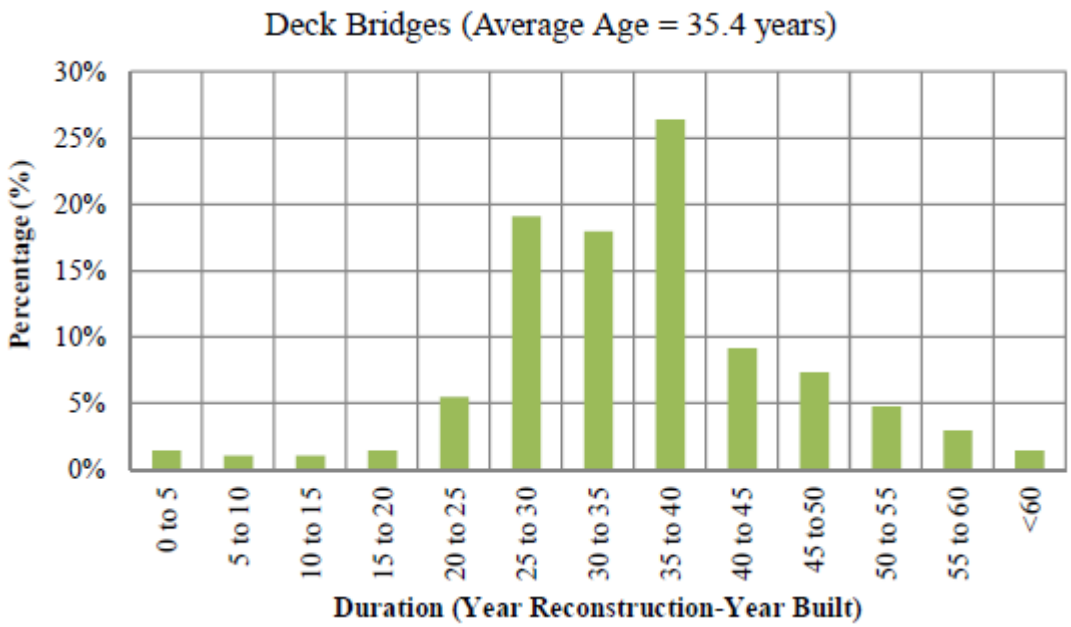
Table 33. Examples of annual maintenance costs.

State	Bridge Type	Annual Maintenance Cost ($/sft)
Iowa	Steel Beam	0.091
	Steel Beam Continuous	0.043
	Prestressed Concrete	0.052
	Reinforced Concrete beam-deck girder	0.082
	Slab	0.062
	Steel High-truss	0.114
	Pony Truss	0.227
Missouri	All	0.093

Source: NCHRP 1986

Bridge structures made using UHPC are projected to last longer than bridges made of conventional reinforced concrete. UHPC is built to withstand the effects of harsh conditions and save money over the life cycle of a project.

Figure 34 shows the service life duration for deck slabs in conventional bridges before replacement. It is observed that most of the state bridges have deck replacement after 25 to 40 years (Morcous 2013).



Source: Morcous 2013

Figure 34. Histogram of deck replacement interval times for state bridges.

4.2.4 Estimating User Costs

User cost estimates are based on time delays, vehicle operation, and crashes (FHWA 2002, Watts et al. 2012). Environmental costs like CO_2 emissions can also be included in the evaluation of new construction materials. User costs can be estimated on a unit cost basis. Table 34 summarizes the user costs units recommended for LCCA.

Table 34. Recommended user cost units for LCCA.

Parameter	Unit
Length of detour	km
Duration of work	Days
Average daily traffic on bridge	Veh/day
Truck percentage on bridge	Percentage
Annual Traffic growth rate on bridge	Percentage
Annual traffic growth rate under bridge	Percentage
Value of time- cars	$/hour
value of time – trucks	$/hour
Vehicle occupancy rate- cars, trucks	$/mile
Estimated travel delay per vehicle on bridge (For bridge replacement, rehabilitation both)	Minutes
Estimated delay under the bridge	Minutes
Cost per accident	$

Source: (Alnewar 2018)

For ABC techniques like Slide in Bridge Construction (SIBC) and Self-Propelled Modular Transport (SPMT) specific considerations for user costs are needed. For example, SBIC user costs can be categorized as follows (Aktan and Attanayake 2017):

- Vehicle operating Cost (VOC)
- Accident Cost (AC)
- Driver Delay Cost (DDC)
- Passenger Cost calculated using Average Vehicle Occupancy (AVO)
- Passenger Delay Cost
- Passenger Accident Cost
- Environmental Cost, Health Care Cost
- Water pollution
- Climate Change cost

Table 35 shows examples of user costs for different parameters that can be included in LCCA. SIBC user costs are compared with conventional construction user costs.

Table 35. SIBC versus conventional construction user costs.

Parameter	SIBC	Conventional Construction
Hourly rate for a truck driver	$24.82/vehicle/hr	$24.82/vehicle/hr
Average hourly vehicle operating cost for trucks	$59.18/Vehicle/hr	$59.18/Vehicle/hr
Normal accident rate for trucks	1.30 Accidents/100 million vehicle miles	1.30 Accidents/100 million vehicle miles
Accident rate per truck-mile	2.30 accidents/100 million vehicle miles	
Volume of Truck traffic on The Roadway impacted during construction	190 vehicles/day	190 vehicles/day

Source: Aktan and Attanayake 2017

The user costs in a work zone can be grouped into two categories: (Ozbay et al. 2003)

- Vehicle operating costs during normal operation and work zone operation.
- Cost of travel delay time during normal operation and work zone operation.

As a reference, FHWA technical bulletin “Life -Cycle Cost Analysis in Pavement Design-Interim Technical Bulletin” describes a twelve-step procedure for calculating user costs (Walls and Smith 1998). The user cost calculation process can also be applied to bridges, and it is summarized as follows:

Step 1 Project future year traffic demand.

Step 2 Calculate work zone directional hourly demand.

Hourly demand can be determined from agency traffic data. If this information is not available, the default hourly distribution from MicroBENCOST can be used.

Step 3 Determine roadway capacity.

Three capacities to be determined from HCM: the free flow capacity of the facility under normal operating condition, the capacity of the facility when the work zone is in place, and the capacity of the facility to dissipate traffic from a standing queue. Capacity during queue dissipation is less than the capacity for free-flow condition. Work zone capacity is estimated based on how many lanes are closed.

Step 4 Identify the user cost components.

Compare the roadway capacity with the hourly demand for the facility.

Step 5 Quantify traffic affected by each cost component.

Quantify the number of vehicles involved with each cost component.

Step 6 Compute reduced speed delay.

To calculate work zone delay, work zone length, speed and upstream speed are needed.

Step 7 Select and assign VOC rates.

Step 8 Select and assign delay cost rates.

Step 9 Assign traffic to vehicle classes.

Step 10 Compute individual user costs components by vehicle class.

Step 11 Sum total work zone user costs.

Step 12 Address circuity and crash costs.

Work zones may have fewer lanes or narrower clearances between vehicles and roadside objects (e.g., owing to decreased or deleted shoulders), reducing capacity. In work zones, speed restrictions may decrease as well (HCM 2010). Temporary road closures may cause traffic to be diverted to other routes, resulting in higher traffic volumes. Therefore, road users experience delays during a construction project because of the following situations:

- Temporary closures of bridge lanes for routine maintenance, repair, and rehabilitation.
- Congestion that develops when such closures slow down traffic and create secondary queuing delays.
- Traffic impending effects of poor roadway conditions.

Travel and Work Zone Delay Costs

When one or more lanes are closed, travel time, and crash rates increase. Drivers may also choose to avoid work zone congestion by taking a detour around the work zone. To calculate travel and work zone delay costs, the number of vehicles and delay per vehicle affected by the construction

must be predicted. Travel time delays for work zone can be calculated from queue length data or spot sensor data (FHWA 2011). The delay per vehicle is calculated with the following equation:

$$\frac{\text{Delay}}{\text{Vehicle}} = L_q\left(\frac{1}{u_q} - \frac{1}{U_{WZSL}}\right) + L_{WZ}\left(\frac{1}{\frac{u_f}{2}} - \frac{1}{U_{WZSL}}\right)$$

Where:

Lq = Speeds over the length of queue

L_{WZ} = Speed at capacity flow

Uq = average speed in queue

L_{WZ} = Speeds over the length of the transition areas of the work zone

U_{WZSL} = Work Zone speed limit

U_f = Free flow speed

The driving factors for detours are height clearance and work zone delays. The vehicles affected are calculated by multiplying the overall ADT by the percentage of the type of vehicle class restricted from travelling over the bridge during construction. Along with the amount of traffic, a growth rate is needed to forecast the vehicle annual demand. Travel and work zone delay costs are calculated with the following equation (NCHRP 2003):

$$TDC_C = \left[tdc_1 v_1 + tdc_2 v_2 + \ldots + tdc_n v_n\right] DT_c$$

Where:

$tdc_1, tdc_2, \ldots\ldots tdc_n$ = delay cost per vehicle per unit time for each vehicle type 1 ,2 ,3…...n

v1, v2, v3 .....vn = number of vehicles of type 1, 2…...n delayed by the action

DT_c = Average delay time per vehicle due to congestion and closure

The final equation to calculate the travel delay cost is:

$$\text{Travel Delay Cost (TDC)} = (ADT \times TDC_c + ADTT \times TDC_T) \times \Delta T$$

Where:

TDC = Total Travel Delay Cost

TDC_c = Travel Delay Cost for Cars ($/hr)

TDC_T = Travel Delay Cost for Trucks ($/hr)

ΔT = Time Delay per Vehicle

ADTT = Number of truck traffic delayed

ADT = Number of Passenger cars delayed

Traffic volumes from 1200 to 1800 vehicles per hour per lane may not have excess queue length beyond the usual traffic congestion in the work zone (NCHRP 2003). If the traffic volume is greater, then queuing models may be needed to estimate travel delays and associated costs.

Vehicle Operating Costs

Vehicle Operating Costs (VOC) may be divided in terms of passenger cars and truck. VOC occur due to the detour of vehicles due to longer road closures, and they depend on cost of fuel. The value of time is expressed as per person hour. VOC are calculated using the following equation (NCHRP 2003):

$$\text{Vehicle Operating Costs (VOC)} = (ADT \times VOC_c + ADTT \times VOC_T) \times \Delta D \quad \ldots\ldots\ldots\ldots\ldots (5)$$

Where:

ADT = Average Daily Traffic (Vehicle Per Day)

VOC_c = Vehicle Operating Cost for Cars ($/vehicle)

ADTT = Average Daily Truck Traffic (Vehicle/day)

VOC_T = Vehicle Operating Cost for Trucks ($/vehicle)

ΔD = Additional Distance Travelled, (mi or km)

Energy Consumption Costs

Energy consumption cost scan be divided into the following:

- Environmental costs due to CO_2 emission from construction machineries.
- CO_2 Emission due to production of concrete materials

Energy consumption cost can be quantified by the amount of CO_2 emissions during construction. The burning of 1-L diesel of fuel per hours is equivalent to 2.62 kg of CO_2 (EPA Greenhouse gases equivalencies calculator). The total consumption of diesel fuel can be converted to the equivalent of CO_2 kg and then to CO_2 Ton.

Environmental impact in terms of CO_2 emissions can be calculated by multiplying the impact of each raw material times the emissions. The unit CO_2 emissions associated with cement, aggregate, steel fiber and water reducing agent are: 0.865 kg-CO2/kg, 0.0013 kg-CO2/kg, 0.94 kg-CO2/kg and 0.0184 kg-CO2/kg (Bouhaya and Roy 2009). Then, CO_2 emissions associated with producing 1 cubic meter of UHPC can be calculated. Different mix designs of UHPC will have different CO_2 emissions. The process is based on the following equation (Dong 2018).

$$Unit\ CO2\ Emission = \sum_{i=1}^{n} ma\ .\ cef$$

Where:

ma = amount of the constituent materials used in the concrete (kg/m^3).

cef = CO2 emission factor associated with manufacture of constituent material (kg-CO2/kg).

n = Number of mixes within the concrete (e.g., UHPC contains steel fibers, cement, silica fume, fine aggregates, water reducing agent, water)

4.2.5 Calculating Net Present Value (NPV) and Equivalent Uniform Annual Cost (EUAC)

Life cycle costs over a specified period are discounted to the present to calculate the Net Present Value (NPV) using the following equation:

$$NPV = CC + \sum_{j=1}^{n1} \frac{MC}{(1+r)^{t2}} + \sum_{k=1}^{n2} \frac{RC}{(1+r)^{t3}} + (-S) or\ (+D) \frac{1}{(1+r)^{N}}$$

Where:

r = Discount rate

CC = Initial construction cost including material and labor cost

MC = Maintenance cost in terms of the agency and user cost of maintenance actions

RC = Rehabilitation cost in terms of the agency and user cost of maintenance actions

S = Salvage value

D = Disposal cost

n1 = Number of maintenance activities over analysis period

n2 = Number of rehabilitation activities over analysis period

N = Length of analysis Period

The life cycle cost of alternatives with different expected life service (e.g. conventional concrete versus UHPC) can be compared using the Equivalent Uniform Annual Cost (EUAC). The EUAC can be computed with the following equation:

$$EUAC = LCC(NPV)\ \frac{r.(1+r)^N}{(1+r)^N-1}$$

Where:

N = Length of the analysis period

r = Discount rate

4.2.6 Deterministic and Probabilistic LCCA

LCCA can be deterministic or probabilistic. Deterministic analysis uses discrete input values and probabilistic analysis accounts for uncertainty and variability in those variables. Cost parameters are modeled with a particular type of distribution (e.g., triangular distribution). The probabilistic analysis provides as an array of costs as an output with their associated probability. A comparison of these two LCCA approaches is shown in Figure 35.

Deterministic LCCA	Probabilistic LCCA
• Discrete Costs • Estimated Average • Acceptable LCC range • Neglects uncertainties	• Probability distribution of cost • Historical data • Probability of component variability • Includes uncertainties • Accounts for inflation

Source: Alipour et al. 2020

Figure 35. Comparison of deterministic and probabilistic LCCA approaches.

Deterministic Analysis

In the deterministic analysis, parameters used in the LCCA are fixed and the agency and user costs over a bridge's life span are added in present value to calculate the NPV. However, a deterministic LCCA does not address the potential range of the values used as data inputs for the parameters involved in the calculations. However, some parameters used in the LCCA cannot be accurately determined (e.g., user costs).

Probabilistic Analysis

In the probabilistic analysis, the LCCA parameters are modeled with distributions instead of using single values. The probabilistic analysis is a risk-based technique that expresses the likelihood of life-cycle variable costs. Life-cycle costs inputs such as material costs, environmental conditions, construction methods, construction duration, and design changes are modeled by probabilistic distributions (NCHRP 2003). Triangular or normal distributions are commonly used in LCCA. Triangular distributions are defined by a mean, upper, and lower values. Normal distributions are defined by a mean and standard deviation. Monte Carlo is a common simulation technique that is used for probabilistic analysis. Table 36 shows probability distributions with the parameters required for the analysis.

Table 36. Examples of probability distribution parameters

Probability Distribution	Parameters
Uniform	Minimum, Maximum
Normal	Mean, Standard Deviation
Log- Normal	Mean, Standard Deviation
Triangular	Minimum, most likely, Maximum
Beta	Alpha, Beta
Geometric	Probability
Truncated Normal	Mean, Standard Deviation, Minimum, Maximum
Truncated Log Normal	Mean, Standard Deviation, Minimum, Maximum

Table 37 shows examples of LCCA user costs parameters for probabilistic analysis. LCCA output values are expressed in a distribution generated by the Monte Carlo simulation.

Table 37. Examples of LCCA user cost parameters for probabilistic analysis.

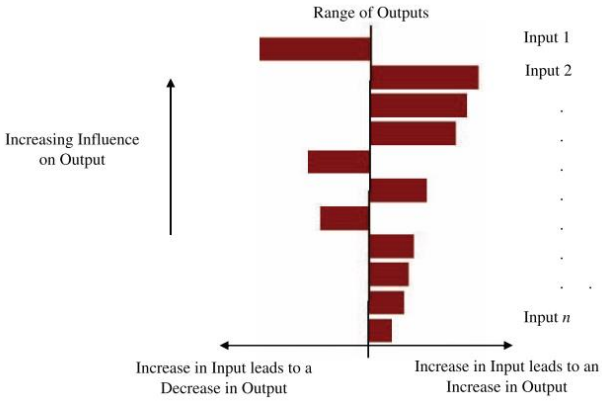
Parameter	Value		
	Mean	Lower	Upper
Personal wage ($ per hr)	21.4	19.6	23.2
Commercial wage ($ per hr)	24.35	22.3	26.4
Vehicle operating cost (VOC) for passenger cars ($per hr)	9.25	8.5	10
VOC for freights ($per hr)	20	18	21.1
Queue length (km)	2	1	2
Accident rate-normal (per million vehicle-km)	215	210	260
Accident rate-during work zone (per million vehicle-km)	240	190	275
Cost per accident ($)	1094	918	1270
Average daily traffic for feature intersected	52086	50000	60000
Average daily traffic for facility carried	15000	13000	17000

Source: Aktan and Attanayake 2015

4.2.7. Sensitivity Analysis

Sensitivity analysis is used to identify input parameters that could have the most impact on the total life-cycle cost. Sensitivity analysis can be conducted by changing one input parameter while all the others remain constant. Changing one parameter at a time is the simplest manner to perform sensitivity analysis. The change in a parameter is defined as a percentage of a reference value.

Tornado plots, spider plots, and elasticity diagrams are representations of the results of sensitivity analysis. These diagrams show output changes when an input variable changes from a minimum to a maximum value while holding all other parameters at their average values (NCHRP 2012). Figure 36 shows an example of a Tornado diagram with the outputs produced by each input variable placed in a descending order of influence starting from the top.



Source: NCHRP 2012

Figure 36. Example of a tornado diagram.

4.3 LCCA Software

LCCA tools to evaluate bridge rehabilitation and replacement strategies are needed to facilitate the calculations in support of the decision-making process. Software tools BridgeLLC and Life 365 are used for LCCA to compare alternatives (NCHRP 2003) (Jaber 2016). Life-365 performs LCCA for bridge components including slabs, beams, and circular columns. Figure 37 shows an example of LCCA results from Life-365.

Life-Cycle Costs

Name	Construction Cost	Barrier Cost	Repair Cost	Life-Cycle Cost
Base case	$3.74 per sq. ft	$0.00 per sq. ft	$23.26 per sq. ft	$27.00 per sq. ft
Alternative 1	$3.74 per sq. ft	$0.00 per sq. ft	$12.18 per sq. ft	$15.91 per sq. ft

Graphs

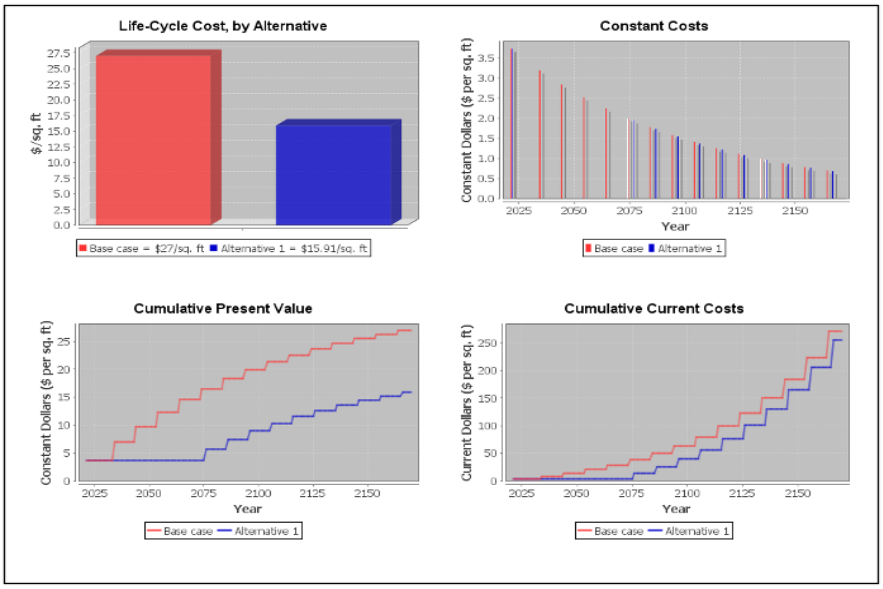


Figure 37. LCCA results using Life 365 software.

The software is based on chloride corrosion models to predict he deterioration of concrete bridge decks. Agency costs are input as cost per square unit, although user costs are not well defined in the software. The chloride diffusion model is useful for assessing the life expectancy of deck overlays and shell encapsulation of columns. Life 365 calculates the present value, and it also identifies the time to scheduled maintenance and rehabilitation activities based on the evolution of damage overt time. Life 365 software can be used to analyze the life expectancy of conventional concrete mixes with reinforced steel subjected to corrosion. Although, a deterioration model for UHPC is not available in Life. Life 365 does not include either user and environmental costs.

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CHAPTER 5: CASE STUDY

This chapter demonstrates with a case study the step-by-step Life Cycle Cost Analysis (LCCA) methodology described in Chapter 4. In this case study, the bridge deck area is 1375 sq. ft. with a slab width of 42 ft. There are two bridge deck retrofitting alternatives under consideration for construction:

Alternative 1: Conventional cast in place concrete deck slab.

Alternative 2: Precast deck slab with UHPC closure joints.

To evaluate the alternatives, LCCA is conducted over a period of 60 years with a discount rate of 3%. Agency costs are based on references provided by the Florida Department of Transportation (FDOT), and user costs are estimated following FHWA guidelines.

5.1 Alternative 1: Conventional Cast in Place Concrete Deck Slab

In this alternative, conventional concrete type II is used for building the in-place bridge deck slab. This type of concrete is recommended when the environmental condition is slightly aggressive. According to the Structure Design Guidelines from the Florida Department of Transportation (FDOT), the environmental condition is classified as slightly aggressive when the chloride content is less than 500 ppm, and the sulphate content is between 150 - 1000 ppm. (FDOT 2022). The main technical properties of type II concrete are shown in Table 38.

Table 38. Properties of type II concrete for alternative 1.

Properties	Typical Values
Air content	1 to 6%
Specified Minimum Strength (28 day)	31 Mpa
Minimum total cementitious materials content	362 Kg/m^3
Maximum water to cement materials ratio	0.44
Maximum Allowable Chloride Content	0.237 Kg/m^3

Source: FDOT 2022

5.1.1 Deterioration Model and Life Expectancy for Alternative 1

The life expectancy of the deck slab is calculated based on the corrosion model. It is assumed that the structure is exposed to chloride with a surface concentration of 10 kg/m3 (moderate for a marine splash zone). The chloride initiation period is calculated and spalling concrete slab damage is projected over time. Corrosion and damage of the conventional concrete is computed with the method described in Chapter 3.

For cast in place (CIP) conventional concrete slab, the spalling damage evolution over time is observed in Figure 38. In this case study, it is considered that the end-of-life service is when the concrete slab reaches 40% or more of damage. Therefore, the expected life for the conventional concrete slab is reached in 30 years.

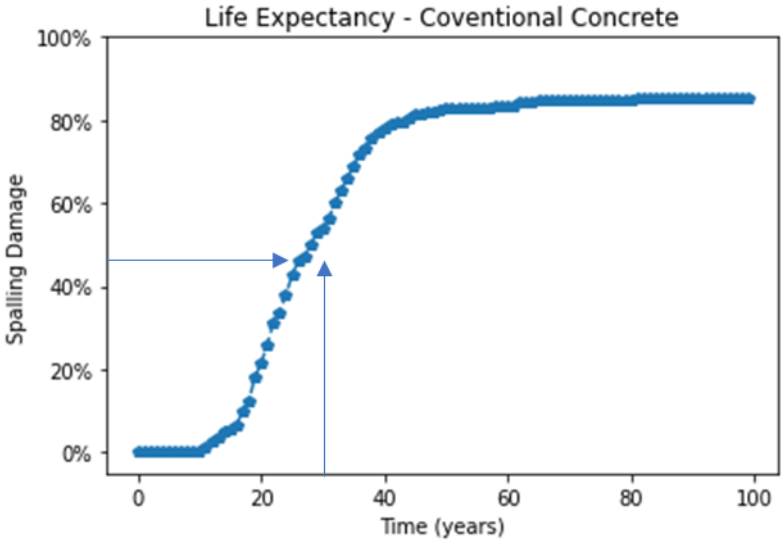


Figure 38. Corrosion initiation and cumulative damage for the conventional concrete slab.

5.1.2 Maintenance and Rehabilitation Activities for Alternative 1

Figure 39 shows a schematic life-cycle activity diagram for alternative 1 with the scheduled maintenance and rehabilitation interventions.

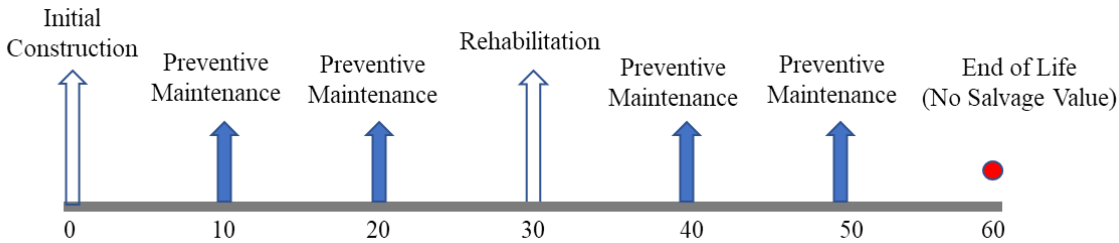


Figure 39. Life cycle activity diagram for alternative 1.

A thin bonded epoxy overlay is scheduled every 10 years as preventive maintenance (Chang et. al. 2016). One disadvantage of epoxy overlays is that they are difficult to repair when they spall or break. The epoxy overlay is removed and replaced as part of bridge maintenance.

The conventional concrete slab has a life expectancy of 30 years. Therefore, rehabilitation with the replacement of the concrete slab scheduled at year 30. A new life cycle for the conventional concrete slab begins and it ends at year 60. At year 60, there is no remaining life neither a salvage value.

5.1.3 Agency Costs for Alternative 1

Table 39 provides a breakdown of initial construction cost estimates for the conventional concrete slab. Cost data are retrieved from the report prepared by FDOT titled "FDOT Bridge Development" with Financial Project ID no 442667-1-22-01 (FDOT 2020).

Table 39. Initial construction cost estimation of cast in place concrete slab, Alternative 1.

Description	Unit	Cost per Unit	Quantity	Cost
Cast in Place Concrete -Class II	Per Cubic Yard	250	1375	$343,750
Reinforcing Steel	Per Pound	1.05	274600	$288,330
Expansion Joint	Per Linear ft	45	42.25	$1901
			Total	$633,981

The cost estimate for a thin bonded epoxy overlay, is $ 22 per sq. ft. (Morcous 2013). Therefore, the cost for a preventive maintenance activity is: 1375 x $22 = $30,250.

The cost of rehabilitation for the replacement of the slab is assumed to be the same as the initial construction cost: $ 633,981.

Table 40 shows the agency costs over the period of analysis with calculations of the present cost using a 3% discount factor.

Table 40. Agency life cycle costs for Alternative 1.

Year	Activities	Agency Costs	Discount Factor (3% Rate)	Present Cost
0	Initial Construction	$633,981	1	$633,981
10	Preventive Maintenance	$30,250	0.744	$22,509
20	Preventive Maintenance	$30,250	0.554	$16,749
30	Rehabilitation (Replacement of the concrete slab)	$633,981	0.412	$261,192
40	Preventive Maintenance	$30,250	0.307	$9,273
50	Preventive Maintenance	$30,250	0.228	$6,900
60	Salvage Value	0		$0
			Total	$950,604

Note: It is assumed in the analysis that there is no inflation.

5.1.4 User Costs for Alternative 1

User costs calculations follows the FHWA method described in Chapter 4 (Walls and Smith 1998). In this method, user cost estimates are based on traffic projections distributed by time periods during the day. In this section, the process to calculate user costs is illustrated for the rehabilitation activity at year 30.

The 24-hours work schedule for the rehabilitation activity is provided in Table 41 with distributed work and non-work zone time periods. The projected Annual Average Daily Traffic (AADT) is assumed to be 114,000 at year 30. From the FHWA research study, the user costs calculation steps are (Walls and Smith 1998) (Ozbay et al 2003):

Step 1 Project future year traffic demand.
Step 2 Calculate Work Zone Directional Hourly Demand.
Step 3 Determine Roadway Capacity.
Step 4 Identify the User Cost Components.
Step 5 Quantify Traffic Affected by Each Cost Component.
Step 6 Compute Reduced Speed Delay.
Step 7 Select and Assign VOC rates.
Step 8 Select and Assign Delay Cost Rates.
Step 9 Assign traffic to vehicle classes.
Step 10 Compute individual user costs components by vehicle class.
Step 11 Sum total work zone user costs.
Step 12 Address circuitry and crash costs.

In this example, traffic delay and vehicle operation costs are included in the calculations. There is no crash data available to calculate crash costs. Default hourly distribution factors generated by MicroBENCOST are used to calculate delay costs and VOC in this case study (Ozbay et al. 2003).

It is worth to mention that work zone directional hourly demand should be calculated from agency traffic records.

Table 41 shows the directional hourly traffic distribution for the inbound and outbound trips. The calculation process is the same for inbound and outbound traffic.

Table 41. Directional hourly traffic distribution.

WZ Status	Time Period	Distribution Factor	Inbound (%)	Outbound (%)	Demand	
					Inbound	Outbound
WZ	12-1	1.2	47	53	752	725
WZ	1-2	0.8	43	57	458	520
WZ	2-3	0.7	46	54	429	431
WZ	3-4	0.5	48	52	320	296
WZ	4-5	0.7	57	43	532	343
WZ	5-6	1.7	58	42	1314	814
Non-WZ	6-7	5.1	63	37	4282	2151
Non-WZ	7-8	7.8	60	40	5700	3557
Non-WZ	8-9	6.3	59	41	4953	2945
Non-WZ	9-10	5.2	55	45	3811	2668
WZ	10-11	4.7	46	54	2881	2893
WZ	11-12	5.3	49	51	3461	3081
WZ	12-13	5.6	50	50	3731	3192
WZ	13-14	5.7	50	50	3798	3249
WZ	14-15	5.9	49	51	3852	3430
Non-WZ	15-16	6.5	46	54	3984	4001
Non-WZ	16-17	7.9	45	55	4737	4953
Non-WZ	17-18	8.5	40	60	4531	5814
Non-WZ	18-19	5.9	46	54	3617	3632
WZ	19-20	3.9	48	52	2495	2312
WZ	20-21	3.3	47	53	2067	1994
WZ	21-22	2.8	47	53	1754	1692
WZ	22-23	2.3	48	52	1471	1363
WZ	23-24	1.7	45	55	1019	1066

Work zone capacities by vehicles per hour are provided in Exhibit 10-14 of the 2010 Highway Capacity Manual (HCM) (HCM 2010). The vehicles per hour are calculated based on the number of lanes open at the work zone. This example assumes that there are three bridge lanes open for traffic with one work zone lane. According to the HCM, the work zone capacity is 1450 veh/hr/lane for this traffic work zone conditions. Non work zone capacity is assumed to be 1900 veh/hr/ln. Lane closures and capacity ranges during construction are described in more detail in Chapter 10 of the 2010 HCM.

Table 42 shows the cost factors associated with work zone activities based on the number of queued vehicles and operating conditions. The delay cost components are speed change delay, total stopping delay, and total queue reduced speed delay. The step-by-step calculation of the delay cost are from FHWA manuals and reports (Walls and Smith 1998) (Ozbay et al 2003).

Table 42. Work zone and non-work zone operating conditions based on vehicle delayed.

WZ Status	Time of day	Future traffic	Capacity	Queue rate	Queued Vehicle	Operating Conditions	Cost Factors
WZ	12AM - 1AM	752	1450	-698	0	Free flow work zone in place no queue	no costs
WZ	1AM-2AM	458	1450	-992	0		
WZ	2AM-3AM	429	1450	-1021	0		
WZ	3AM-4AM	320	1450	-1130	0		
WZ	4AM-5AM	532	1450	-918	0		
WZ	5AM-6AM	1314	1450	-136	0		
Non-WZ	6AM-7AM	4282	5700	-1418	0	No work zone, free flow	no costs
Non-WZ	7AM-8AM	5700	5700	0	0		
Non-WZ	8AM-9AM	4953	5700	-747	0		
Non-WZ	9AM-10AM	3811	5700	-1889	0		
WZ	10AM-11AM	2881	1450	1431	1431	Forced Flow, WZ in place	Speed change VOC and Delay Cost
WZ	11AM-12PM	3461	1450	2011	3442		Stopping VOC and Delay Cost
WZ	12PM-1PM	3731	1450	2281	5723		Total idling VOC or queue reduced speed delay
WZ	1PM-2PM	3798	1450	2348	8071		
WZ	2PM-3PM	3852	1450	2402	10473		
Non-WZ	3PM-4PM	3984	5700	-1716	8758	Forced Flow, WZ in place	Speed change VOC and Delay Cost
Non-WZ	4PM-5PM	4737	5700	-963	7795		Stopping VOC and Delay Cost
Non-WZ	5PM-6PM	4531	5700	-1169	6626		
Non-WZ	6PM-7PM	3617	5700	-2083	4542		Total idling VOC or queue reduced speed delay
WZ	7PM-8PM	2495	1450	1045	5587	Forced Flow, WZ in place	Speed change VOC and Delay Cost
WZ	8PM-9PM	2067	1450	617	6204		Stopping VOC and Delay Cost
WZ	9PM-10PM	1754	1450	304	6507		
WZ	10PM-11PM	1471	1450	21	6529		Total idling VOC or queue reduced speed delay
WZ	11PM-12AM	1019	1450	-431	6098		

Based on the work zone hour that the queue occurred, the following costs are calculated:

- Speed change VOC and delay cost.
- Stopping VOC and delay cost.
- Total idling VOC or queue reduced speed delay cost.

These costs are incurred by vehicles traversing the work zone, traversing queue, stopping during the queue, or slowing down due to the queue. Table 43 shows the vehicles traversing the work zone, stopping, and slowing down.

Table 44 shows the calculation of the average queue delay per vehicle (hour). Queued vehicles are obtained from vehicles traversing the queue at work zone time periods from 10 to 15,15 to 19, and 19 to 24.

From the FHWA reference, the value of time ($/hr) is $11.58 for passenger cars and $20.43 for trucks (Walls and Smith 1998). The case study uses the cost or value of time per vehicle directly from the FHWA reference assuming that there is no inflation. Again, the costs or value of time per vehicle should be updated by the agency for each specific project.

Table 43. Calculation of vehicles traversing, stopping, or slowing down on queue.

Future traffic	Capacity	Queue rate	Queued Vehicles	Number of Vehicles that				
				Traverse WZ	Traverse Queue	Stop	Slowdown	Average Vehicle
752	1450	-698	0	752	0	0	752	0
458	1450	-992	0	458	0	0	458	0
429	1450	-1021	0	429	0	0	429	0
320	1450	-1130	0	320	0	0	320	0
532	1450	-918	0	532	0	0	532	0
1314	1450	-136	0	1314	0	0	1314	0
4282	5700	-1418	0	0	0	0	0	0
5700	5700	0	0	0	0	0	0	0
4953	5700	-747	0	0	0	0	0	0
3811	5700	-1889	0	0	0	0	0	0
2881	1450	1431	1431	1450	1450	2881	0	(0+1450)/ 2 = 716
3461	1450	2011	3442	1450	1450	3461	0	2436
3731	1450	2281	5723	1450	1450	3731	0	4582
3798	1450	2348	8071	1450	1450	3798	0	6897
3852	1450	2402	10473	1450	1450	3852	0	9272
3984	5700	-1716	8758	0	5700	3984	0	9615
4737	5700	-963	7795	0	5700	4737	0	8276
4531	5700	-1169	6626	0	5700	4531	0	7210
3617	5700	-2083	4542	0	5700	3617	0	5584
2495	1450	1045	5587	1450	1450	2495	0	5065
2067	1450	617	6204	1450	1450	2067	0	5895
1754	1450	304	6507	1450	1450	1754	0	6356
1471	1450	21	6529	1450	1450	1471	0	6518
1019	1450	-431	6098	1019	1450	1019	0	6313
			Total	17874	37300	43398	3805	

Table 44. Calculation of average queue delay per vehicle.

Time	Queued Vehicles	Average queue vehicles (a)	Average Queue Length (Miles) b= (a/4*40/5280)	Time @ Queue Speed c=b/8	Time @ free flow Speed d= b/65	Average Queue Delay per Vehicle (hour) (c-d)
10-15	7250	4780.63	9.05	1.13	0.14	0.99
15-19	22800	7991.65	15.14	1.89	0.23	1.66
19-24	7250	6029.40	11.42	1.43	0.18	1.25

Speed change VOC and delay costs are calculated from slowed down vehicles. VOC and added travel time per 1000 vehicles are fixed values adopted from a FHWA research study (Ozbay et al. 2003). VOC and added travel time calculations should be updated periodically. The case study used the values from the FHWA reference assuming that there is no inflation over time. In this case study, speed change VOC and added travel time per 1000 vehicles are calculated for change of speed from 55 mi/hr to 40 mi/hr.

The stopping VOC and delay costs are calculated from stopped vehicles. It is noted that the truck percentage is assumed to be 1.55%. The idling VOC is calculated for the three work zone time periods and added to the user costs.

The following numerical example illustrates the VOC calculation process:

Idling VOC for Passenger Car = Queued vehs x (1-0.155) *Added VOC cost/1000 veh x Average queue delay per vehicle

= 7250 x (1-0.155) x 0.69 x 0.99 + 22800 x (1-0.155) x 0.69 x 1.66+7250 x (1-0.155) x 0.69 x 1.25

= $31536 per day

Total Idling VOC for passenger car in 14 days: $31536 /per day x 14 days = $441,504.

Queue speed delay cost is calculated using the following formula:

Queue Speed Delay Cost = Queued Vehicles x (1-0.155) x 11.58 x Average Queue Delay per Vehicle (hour)

Table 45 shows the detailed calculation of VOC and delay costs.

Table 45. Breakdown calculation of VOC and Delay costs.

A. Speed change VOC and Delay Cost					
Inbound	**Affected vehicles (a)**	**VOC cost/1000 veh (b)**	**Speed change VOC (c = a/1000*b*14)**	**Added travel time (hour/1000vehs) d**	**Speed change Delay cost e = d*a/1000*x*14)**
Pass car	3215	87.28	3928	4.29	$2,236
Truck	590	147.44	1217	6.60	1$,113
Total			$5,145	Total	$3,349
B. Stopping VOC and Delay cost					
Inbound	**Affected vehicles**	**VOC cost/1000 veh**	**Stopping VOC**	**Added travel time hour/1000vehs**	**Stopping Delay cost**
Pass car	36671	109.02	55970	6.78	$40,308
Truck	6727	195.84	18442	9.53	$18,335
Total			$74,414	Total	$58,643
C. Idling VOC and Queue reduced delay cost					
Inbound	**Affected vehicles**	**VOC cost/1000 veh**	**Idling VOC**		**Queue Speed Delay cost**
Pass car	36671	0.69	$441,504		$7,410,986
Truck	6726	0.77	$90,169		$2,398,340
Total			$533,484		$9,809,326

Value x, for passenger Car = $11.58, For Truck = $20.43

Total VOC for inbound vehicles: Speed Change VOC + Stopping VOC + Idling VOC

Total VOC for inbound vehicles = $5,145 + $,74,414+$533,485 = $613,044.

The calculation process was shown in detail for the inbound trips. A similar calculation process should be followed for the outbound trips. In this case study, there are not calculations for outbound trips and the same assumption is made for both alternatives 1 and 2.

Total Delay Cost = Speed Change Delay Cost + Stopping Delay Cost + Queue Speed Delay Cost

= $3,349 + $58,644 + $9,809,327 = $9,871,320.

Total User Costs at year 30 = Total VOC for inbound vehicles + Total Delay Cost

Total User Costs at year 30 = $613,044 + $9,871,320 = $10,484,364.
Present User Cost at year 0 = (1/ (1+0.03) ^30) x 1048364 = $4,319,557

5.1.5 Net Present Value and Equivalent Uniform Annual Costs for Alternative 1

Table 46 shows the Net Present Value (NPV) broken down by agency and user costs and the corresponding Equivalent Uniform Annual Cost (EUAC) for alternative 1.

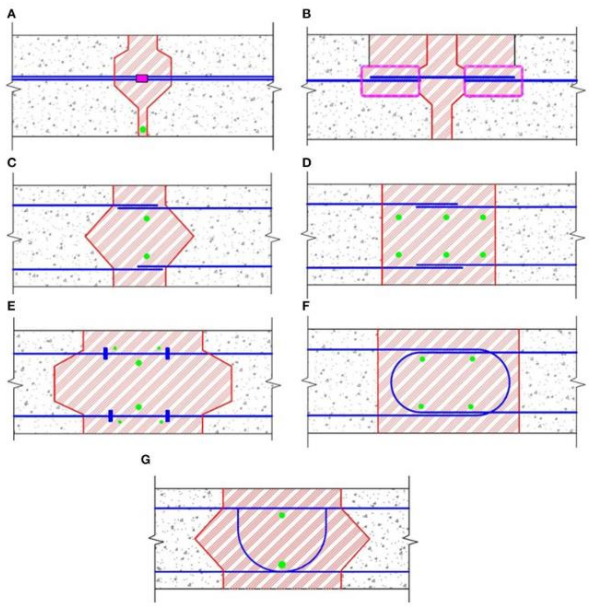
Table 46. Net Present Value and Equivalent Uniform Annual Cost, Alternative 1.

Cost	Alternative 1 Cast in place concrete slab
Agency Cost	$950,604
User Cost	$4,319,557
Total Net Present Value	$5,270,161
Equivalent Uniform Annual Cost	$190,427

The total user cost is about four times than the total agency cost. This is an indication of the importance of including user costs in the LCCA. In this example, user costs were only calculated for the rehabilitation activity which is the replacement of the concrete slab at year 30. If user costs for the initial construction had been also considered, the total user costs would have been higher.

5.2 Alternative 2: Precast deck slab with UHPC closure joints.

Alternative 2 is a prefabricated deck slab with UHPC closure joints and an overlay to protect the deck surface. Common closure joint types used for ABC techniques are shown in Figure 40 (Jaberi Jahromi et al. 2020). The joint configuration for the project corresponds to Figure 40 C. The prefabricated bridge deck is also made with UHPC. Construction can be completed in 4 days at the site according to FDOT information.



Source: Jaberi Jahromi et al. 2020

Figure 40. Schematic configurations for common closure joints in ABC. (A) posttensioning, (B) mechanical connectors, (C) ultra-high performance with straight bars, (D) normal-strength concrete with straight bars, (E) normal-strength concrete with headed bars, (F) normal-strength concrete with 180◦ hooked bar, (G) normal-strength concrete with 90◦ hooked bar.

A field-cast lap joint benefits from UHPC when it serves as a connector between precast bridge deck panels. The rebar of a panel overlaps with the rebar of the adjacent panel, and the gap is filled with UHPC. The connection has short, straight rebar spliced at the same intervals as traditional deck construction to transfer moment, shear, and tensile forces across the joint. However, UHPC has a longer lifespan than conventional concrete while reducing the splice in closure joints (AASHTO 2018). The UHPC mix design reference is from a research study conducted in 2021 for nonproprietary UHPC transverse field joints (Abokifa et al. 2021). The mix design and main mechanical properties of the UHPC mix are presented in Table 47.

Table 47. UHPC mix design properties for closure joint.

Material	1% steel Fiber Mix Quantity kg/m3
Cement	707
Slag	354
Silica Fumes	118
Water	236
w/b	0.2
Sand	1179
Steel Fibers	79
Superplasticizer	13.5
Properties	**1% Steel Fiber Mix**
Compressive Strength, Mpa	128.9
Flexural Strength, Mpa	14.3
Direct Tensile Strength, Mpa	4.83
Water Cement Ratio	0.25

Source: Abokifa et al. 2021

UHPC is tailored specifically for ABC construction since it has very high early strength of 55MPa in 12 hours at normal ambient conditions (Abokifa et al. 2021).

5.2.1 Deterioration Model and Life Expectancy for Alternative 2

The life expectancy of the precast deck slab with UHPC is calculated with the corrosion model described in Chapter 3. It is assumed that the bridge is exposed to chloride with a surface concentration of 10 kg/m3 (moderate for a marine splash zone). The difference with the deterioration model in alternative 1 is in the parameters used the equations for the concrete properties (e.g., diffusion coefficient, chloride threshold value). The corrosion initiation time is longer for UHPC, although chloride propagation still causes concrete spalling damage as shown in Figure 41.

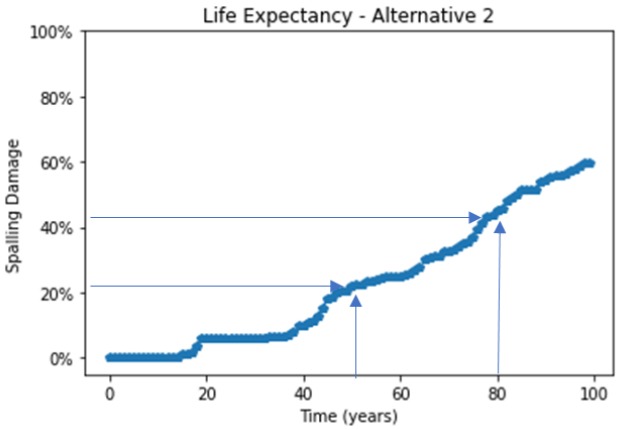


Figure 41. Corrosion initiation time and cumulative damage for the UHPC slab.

It is observed that 40% of deck spalling damage of the slab is expected to be reached in 80 years. There is no full slab deck replacement in this alternative since the analysis period for the LCC is 60 years. However, the overlay to protect the deck surface reaches 20% of spalling damage at year 50 and its replacement is scheduled for rehabilitation.

5.2.2 Maintenance and Rehabilitation Activities for Alternative 2.

Figure 42 shows a schematic life-cycle activity diagram for alternative 2 with the scheduled maintenance and rehabilitation interventions.

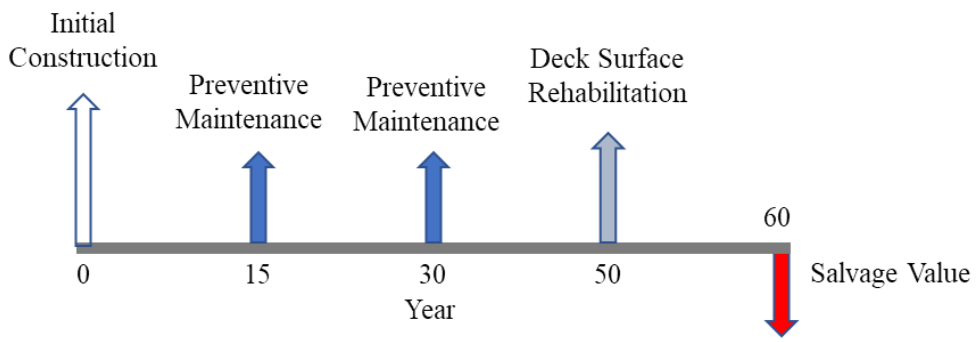


Figure 42. Life cycle activity diagram for alternative 2.

UHPC overlay repair in 10% of the deck area (1375 * 10% = 137.5 sq. ft.) is scheduled every 15 years as preventive maintenance. At year 50, a 1-inch overlay is scheduled for deck surface rehabilitation. At year 60, there are 20 years of remaining life for the precast deck slab since the service life is 80 years.

5.2.3 Agency Costs for Alternative 2

The initial construction cost was from a FDOT project that consist of an approach slab replacement with a construction area of 1375 sq. ft. This project was managed by FDOT District Three (Project Name: I-10 (SR 8) over CR-268A, Bridge No. 500080 FPID: 445645-1). In this project, the deck slab with UHPC closure joints has an initial construction cost of $980,000.

The unit cost reference for the UHPC overlay is from the project BR 1-438 on N463 Blackbird Station Road Over Blackbird Creek stored in the ABC-UTC database. Form this project, the UHPC unit cost for an overlay is $375 per cubic ft. Therefore, the unit cost for the 1-inch UHPC overlay will be: $375 x 0.083 = 31 $/sq-feet (Note: 1 inch = 0.083 ft).

For preventive maintenance, it is assumed that 10% of the deck will need overlay repair every 15 years. Therefore, the cost per preventive maintenance activity is: 31 x 0.10 x 137.5 sq-feet = $4,263.

The of rehabilitation for the replacement of the overlay to protect the deck surface at year 50 is: 31 x 1375 sq-feet = $42,625.

At year 60, the bridge slab deck has 20 years of remaining life and consequently a salvage value. There is no consensus on how to estimate the salvage value. One approach is to account for the costs of demolition and removal while considering the recycled value of the material waste. Another approach seeks the relative value of the serviceability with respect to cost of rehabilitation. In this case study, the remaining life of the pre-cast slab is 25% of the total expected service life of 80 years. Therefore, the salvage value is estimated as 25% of the initial construction cost.

Salvage value at year 60: 980,000 x 25% = $245,000.

Table 48 shows the agency costs over the period of analysis with calculations of the present cost using a discount rate of 3 %.

Table 48. Agency life cycle costs for alternative 2.

Year	Activities	Agency Costs	Discount Factor (3% Rate)	Present Cost
0	Initial Construction	$980,000	1	$980,000
15	Preventive Maintenance	$4,263	0.642	$2,736
30	Preventive Maintenance	$4,263	0.412	$1,756
50	Rehabilitation	$42,625	0.249	$9,723
60	Salvage Value	- $245,000	0.170	-$41,585
`			Total	$952,630

5.2.4 User Costs for Alternative 2

User costs calculations for alternative 2 follow similar steps and assumptions as alternative 1. The difference is that the construction work duration is 4 days instead of 14 days. For user cost calculations, it is assumed that the working hours in alternative 2 are the same as alternative 1. In practice, this assumption should be reviewed for specific project conditions since ABC projects may require a different work schedule than conventional construction practices. Only inbound trips are considered in the user cost calculations. Following the user cost calculation process explained in detail for alternative 1, the total VOC and delay costs for alternative 2 are summarized as follows:

Total VOC for alternative 2 = $175,155.

Total Delay Costs for alternative 2 = 2,820,377.

Total Present User Costs = ($175,155+$2,820,377) x (1/ (1+0.03) ^47) = $746,662.

5.2.5 Net Present Value and Equivalent Uniform Annual Costs for Alternative 2

Table 49 shows the total Net Present Value (NPV) broken down by agency and user costs and the corresponding Equivalent Uniform Annual Cost (EUAC) for alternative 2.

Table 49. Net Present Value and Equivalent Uniform Annual Costs, Alternative 2.

Cost	Alternative 2 Precast Deck Slab UHPC Joints
Agency Cost	$953,516
User Cost	$746,662
Total Net Present Value	$170,0178
Equivalent Annual Cost	$61,432

5.3 Review of LCCA Results

An overview of the LCCA results for the two alternatives is shown in Table 50.

Table 50. Overview of LCCA results for alternatives 1 and 2.

Cost	Alternative 1 (Cast in place slab with conventional concrete)	Alternative 2 (Precast Deck Slab with UHPC Joints and Overlay)
Agency Cost	$950,604	$952,630
User Cost	$4,319,557	$746,662
Total Net Present Value (NPV)	$5,270,161	$1,700,178
Equivalent Annual Cost	$190,427	$61,432

Figures 43 and 44 show the bridge life-cycle projected cost expenditures over the period of analysis for alternatives 1 and 2. The total NPV for alternative 1 is about three times higher than the total NPV for alternative 2. Although, the agency cost in alternative 1 with conventional concrete is higher than alternative 2 with UHPC, the total NPV of alternative 1 with conventional concrete is higher in the long-term. The final total net present value and equivalent uniform annual cost are lower in alternative 2 with UHPC because of the user costs.

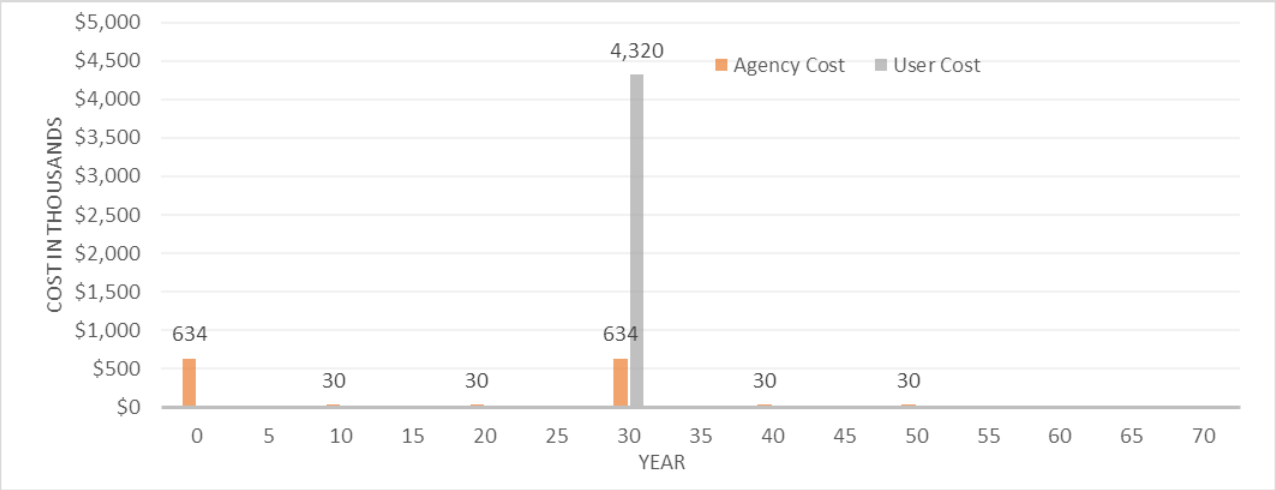


Figure 43. Projected cost expenditures over time, Alternative 1.

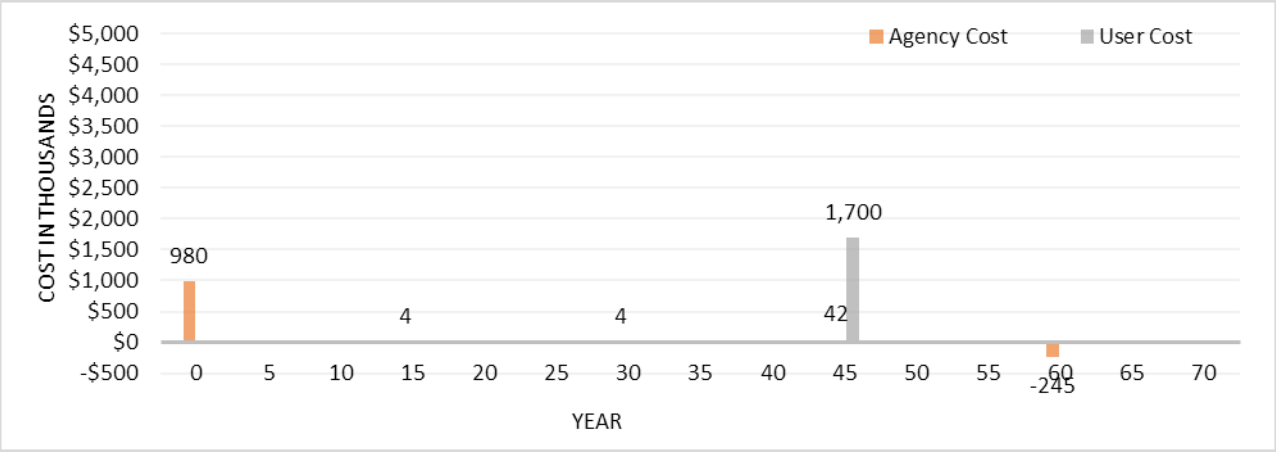
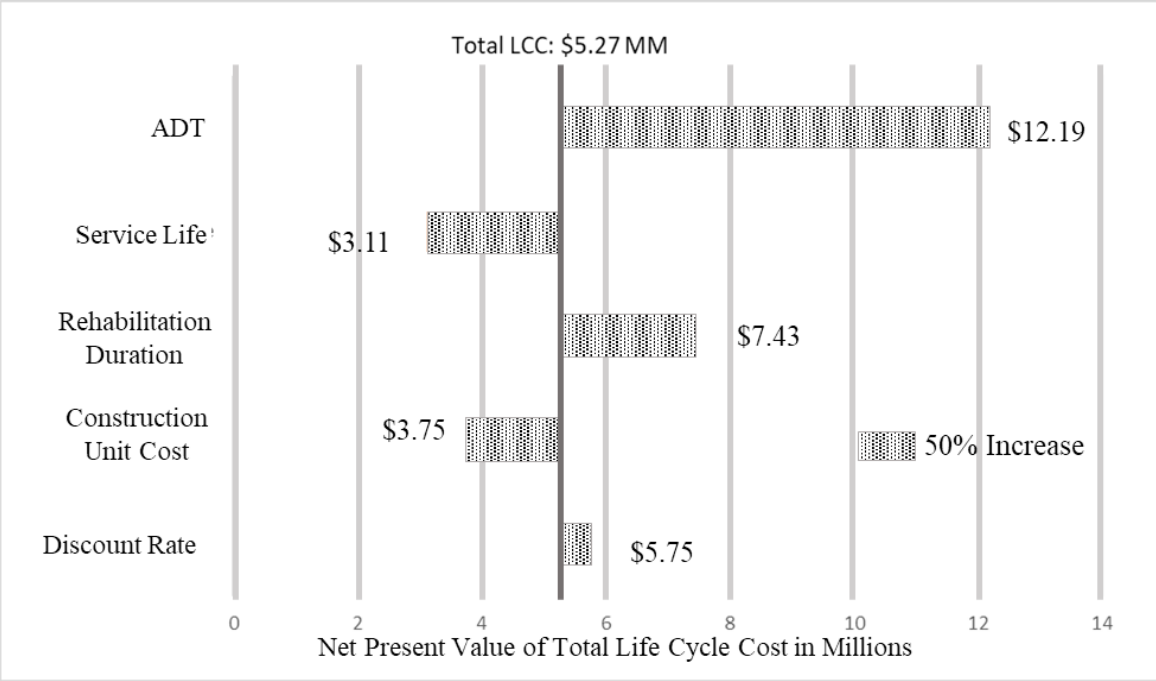


Figure 44. Projected cost expenditures over time, Alternative 2.

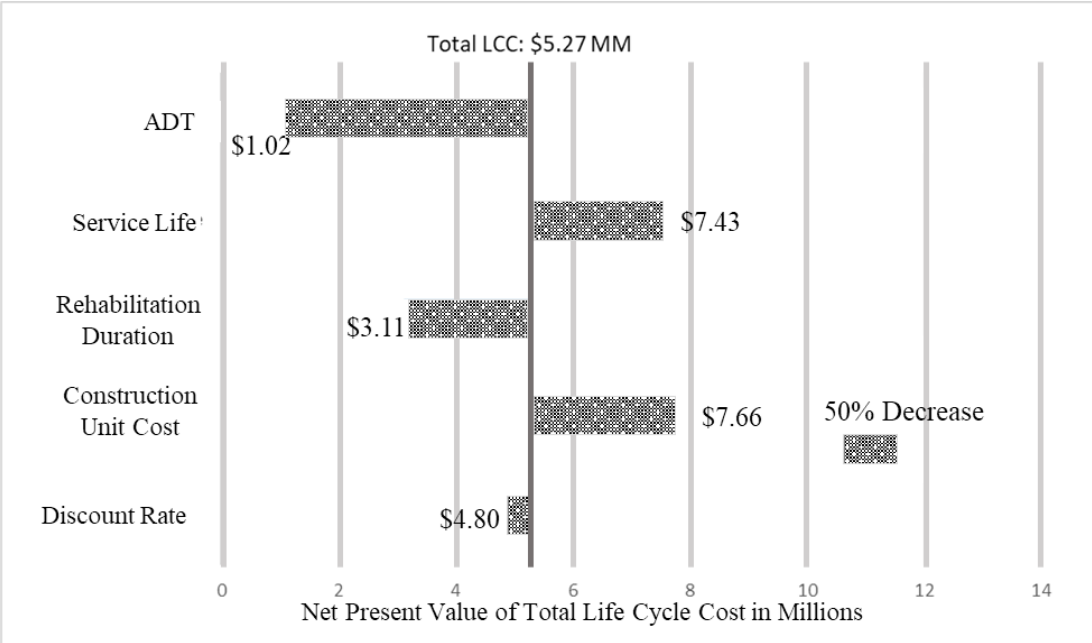
5.4 Sensitivity Analysis

Figure 45 shows the tornado diagram with the results of a life cycle cost sensitivity analysis for alternative 1. Each individual variable was varied 50% while the other variables remain constant. The construction unit cost, average daily traffic, rehabilitation duration, and discount rate values were varied positive 50% and negative 50% in the sensitivity analysis. The sensitivity analysis was performed to identify the most relevant factors that influence the life cycle costs. It also provides insights of the best and worst-case scenarios. Figure 45 (a) shows the effect of 50% increase of the individual data input variables in the life cycle costs and Figure 45 (b) the effect of 50% decrease.

Figure 45 (a) shows that a 50% increase of service life significantly decreases the total life cycle costs. The opposite is observed in Figure 45(b). Whereas if the duration of rehabilitation increases by 50%, the total life cycle cost increase significantly. Decreasing the duration of rehabilitation decreases the total life cycle costs.



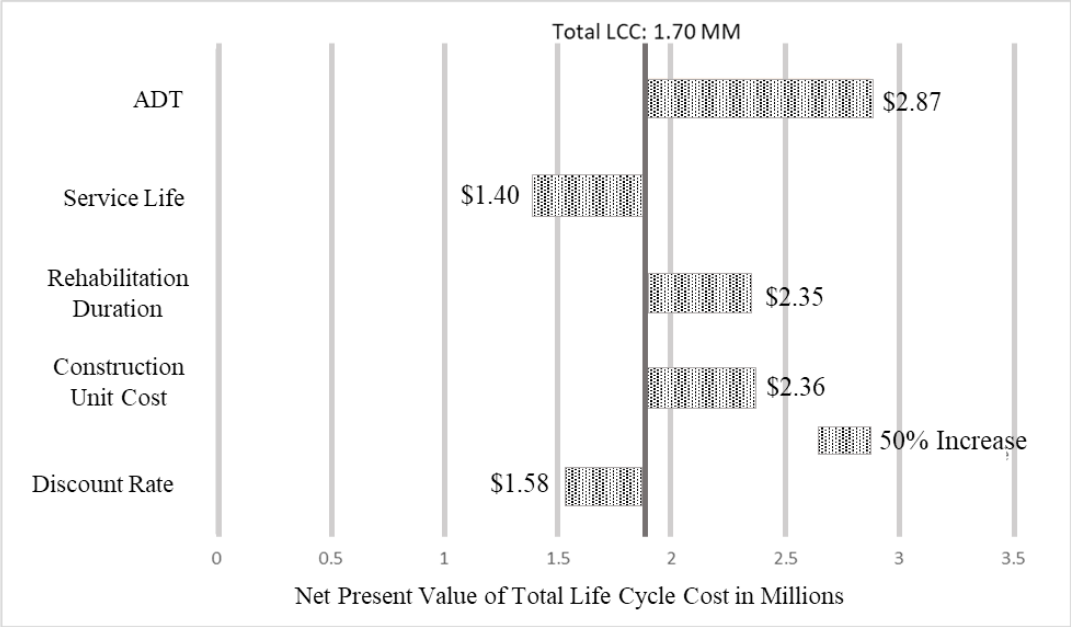
(a) 50% increase of LCCA data input variables



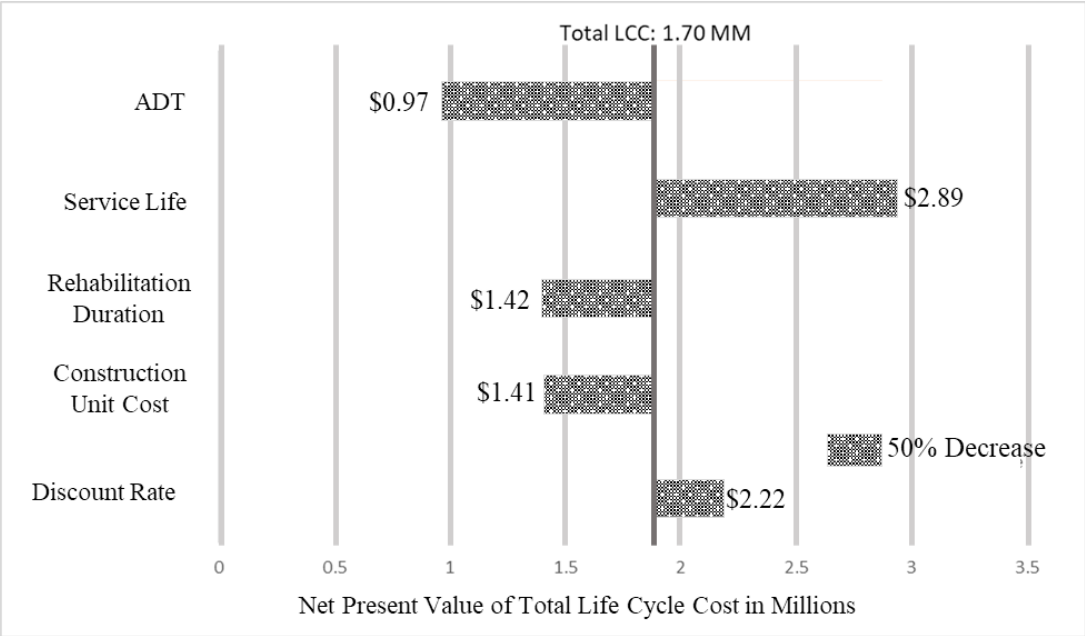
(b) 50% decrease of LCCA data input variables

Figure 45. Life cycle cost sensitivity analysis tornado diagram, Alternative 1.

Figure 46 shows the tornado diagram with the results of the life cycle cost sensitivity analysis for alternative 2. It is observed that the total life cycle cost for alternative 2 is less sensitive to ADT variations when compared to alternative 1. These results are influenced by the difference in construction days between alternatives 1 and 2 (14 days versus 4 days). For this reason, the user cost/agency cost ratio in alternative 1 is higher than in alternative 2 (4.54 versus 0.78). It is also observed that that total life cycle cost is more sensitive to construction unit cost variations in alternative 2.



(a) 50% increase of LCCA data input variables



(b) 50% decrease of LCCA data input variables

Figure 46. Life cycle cost sensitivity analysis tornado diagram, Alternative 2.

The results of the sensitivity analysis show that in both alternatives the Average Daily Traffic (ADT) has the most significant effect on the total life cycle cost.

5.5 Recommendation based on LCCA Results

The agency cost is about the same for both alternatives. In alternative 1 with conventional concrete, the agency cost is slightly lower than alternative 2 with UHPC. However, the different frequency of maintenance activities and cost influence the agency costs. The lower frequency of preventive maintenance in alternative 2 due to the higher durability of UHPC is reflected in the results balancing the initial construction cost. At the end of the 60-year analysis period, the total agency cost of the alternatives is very close ($950,604 in alternative 1 versus $952,630 in alternative 2).

The user cost is lower for alternative 2 because the construction time is lower than alternative 1 (4 days versus 14 days). ABC projects have higher initial construction costs, however, there are time savings due to shorter construction times that are reflected in the user costs. When user costs are included in the analysis, the total life-cycle cost of alternative 1 – including agency and user costs - is about three times the total life cost of alternative 2 ($ 5,270,161 versus $1,700,178). Therefore, alternative 2 with UHPC is recommended as the most cost-effective solution in the case study.

It is also concluded that the concrete life expectancy affects agency and user costs over the lifetime of a bridge element. The life expectancy of the precast deck slab with UHPC was almost twice than conventional cast in place concrete deck slack. and this difference is reflected in the LCCA results.

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CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

A Life Cycle Cost Analysis (LCCA) approach has been developed in this research study to quantify the potential benefits of UHPC applications in retrofitting techniques. From a technical perspective, UHPC is an alternative to conventional concrete as a repair material. To choose a retrofit material, failure bridge conditions and expected performance should be analyzed. Common types of failure for concrete bridge elements include leakage, settlement, deflection, wearing, spalling, disintegration, and cracking. One major cause of failure is steel corrosion of reinforced concrete caused by chloride. This cause of concrete failure is common on bridges located near the ocean or from application of de-icing salt. UHPC can potentially result in technical advantages by decreasing steel corrosion deterioration rate due to its low permeability that functions as a barrier to chloride. Despite the technical potential advantages for UHPC as a repair material, the initial cost is much higher than conventional concrete. A LCCA is needed to evaluate both alternatives to identify cost-effective long-term maintenance strategies for ABC projects.

The LCCA performance-based material approach developed in this research includes: (a) a framework for pre-selecting concrete repair material, (b) life expectancy performance models, and (c) a step-by-step methodology to compare life-cycle costs of UHPC to conventional concrete (CSC) in retrofitting techniques. Products delivered from this research aim to support decisions at the network and project management levels.

6.1 Framework for Pre-Selecting Concrete Repair Material

A framework to evaluate alternatives is used for pre-selecting concrete repair material based on expected performance. Durability, mechanical properties, and construction factors are considered in the evaluation of concrete performance. The performance-based framework consists of seven steps:

Step 1: Evaluate the bridge damage to determine the cause of failure.
Step 2: Assess concrete substrate.
Step 3: Identify the retrofitting techniques to repair the damage.
Step 4: Establish performance criteria for repair material.
Step 5: Propose alternative repair materials in agreement with the retrofitting technique.
Step 6: Evaluate expected performance of the alternative repair materials and compare with the design targets.
Step 7: Conduct Life-Cycle Cost Analysis (LCCA) to select the most cost-effective material.

This performance-based framework is a tool to identify cost-effective concrete materials and retrofitting techniques to repair bridge damage. Critical factors in the material selection process are: (a) life expectancy, (b) costs of initial construction, (c) cost and timing of maintenance and rehabilitation activities.

6.2 Life-expectancy Performance Models for UHPC Retrofitting Techniques

Deterministic and probabilistic life-expectancy modeling approaches were analyzed in this research for UHPC elements. The Long-Term Bridge Performance (LTBP) program by FHWA developed a life expectancy model for UHPC elements using deep learning techniques. Due to limited condition data available to model UHPC performance, a hybrid approach that combines Markov chain with time or state-based condition with existing corrosion deterministic models were explored in this research. An additional challenge faced in the development of the UHPC life-expectancy model was how to consider the interaction among bridge components and its effects on structural bridge integrity.

A chloride corrosion deterioration model was finally selected in this research to consider the influence of concrete properties in the expected performance. The reinforced concrete deterioration

model is based on chloride corrosion activity. Concrete materials have critical chloride content thresholds that varies with the bridge site location and concrete mix composition. Diffusivity of chloride content depends on the concrete mix properties that affects the expected corrosion cracking time.

Life expectancy models for UHPC and conventional concrete differ in the chloride diffusion coefficient, water cement ratio, and crack diffusivity. The corrosion initiation period is longer in UHPC reinforced concrete elements than in conventional concrete elements. The derating factors and crack diffusivity result into spalling damage over time in concrete bridges. As shown in Figure 47, the life expectancy model predicts that 40% spalling damage is expected for conventional concrete after 30 years, and the same amount of spalling damage is projected for UHPC after 80 years.

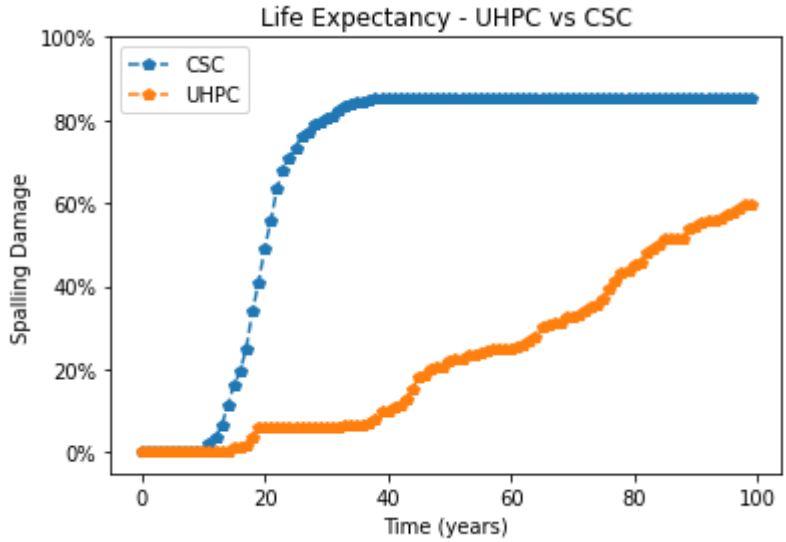


Figure 47. Spalling damage and life expectancy of CSC and UHPC.

6.3 Life Cycle Costs Comparison of Conventional Concrete and UHPC Alternatives for Bridge Maintenance and Rehabilitation.

UHPC is used in both preventive maintenance and rehabilitation techniques for ABC projects. A wide variety of UHPC mix designs have been developed for retrofitting applications in bridge elements. Examples of UHPC retrofitting applications are column shell encapsulation, deck closure joints, thin deck overlay, and link slab connections. Other examples are box girder, bulb T girder, I girder, full depth waffle deck panel, precast pile for deep foundation, precast cantilever retaining walls.

Engineering examples have demonstrated that UHPC enables precast bridges to be lighter in weight, higher in strength, and longer in spans. For example, prefabricated UHPC girders display superior shear and flexure capacity that results in smaller girder sections, higher span-to-depth ratio, and increased girder's centerline spacing. The enhanced durability properties of UHPC aims to extend the overall bridge service life. UHPC longer life-expectancy and frequency of maintenance and rehabilitation interventions influence the LCCA results. Environmental exposure conditions, construction time, and traffic volume are also among the major factors that influence LCCA.

The initial construction cost of ABC projects with UHPC may be higher than projects with conventional concrete, although maintenance and rehabilitation interventions could balance this difference over time, especially when user costs are included in the analysis. Therefore, it is important to consider agency and user costs in LCCA.

Figure 48 shows the comparison of agency and user costs between the two alternatives under consideration in the case study presented in Chapter 5.

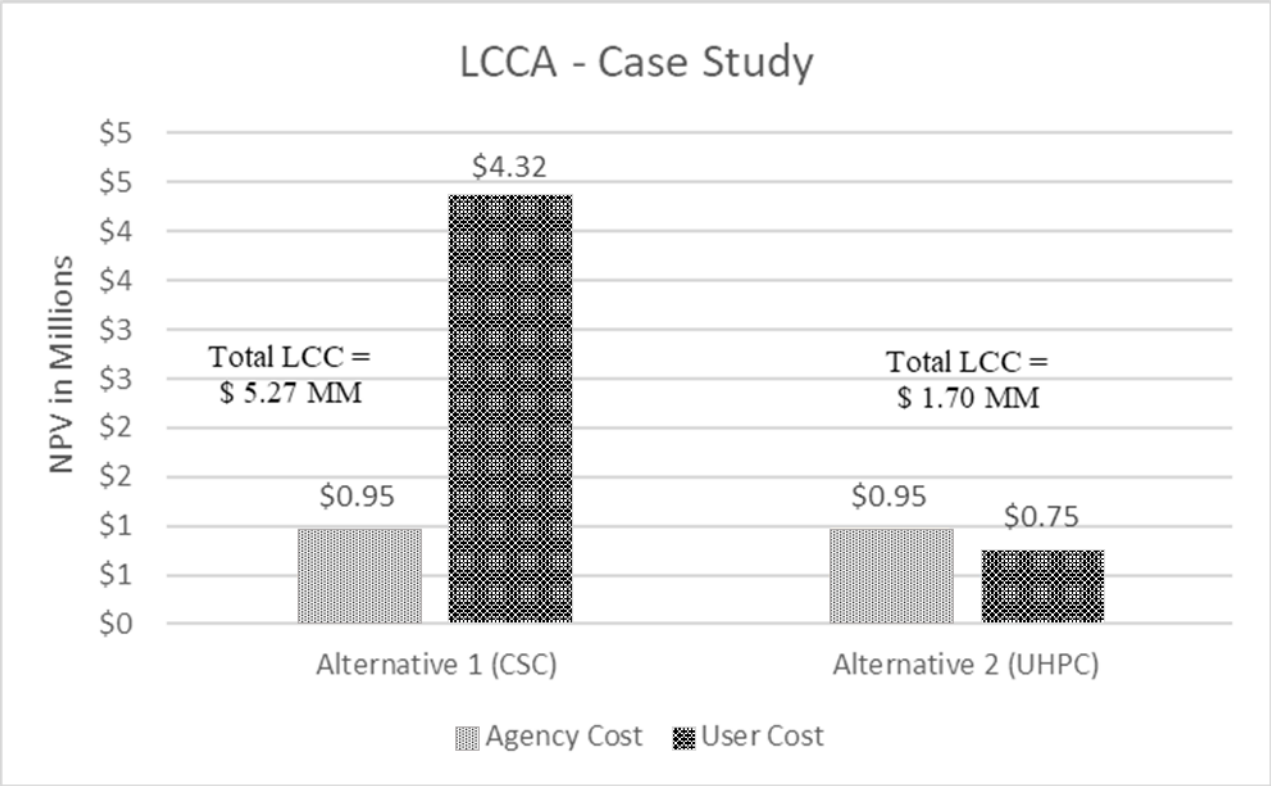


Figure 48. Comparison of life cycle costs for CSC and UHPC retrofitting alternatives.

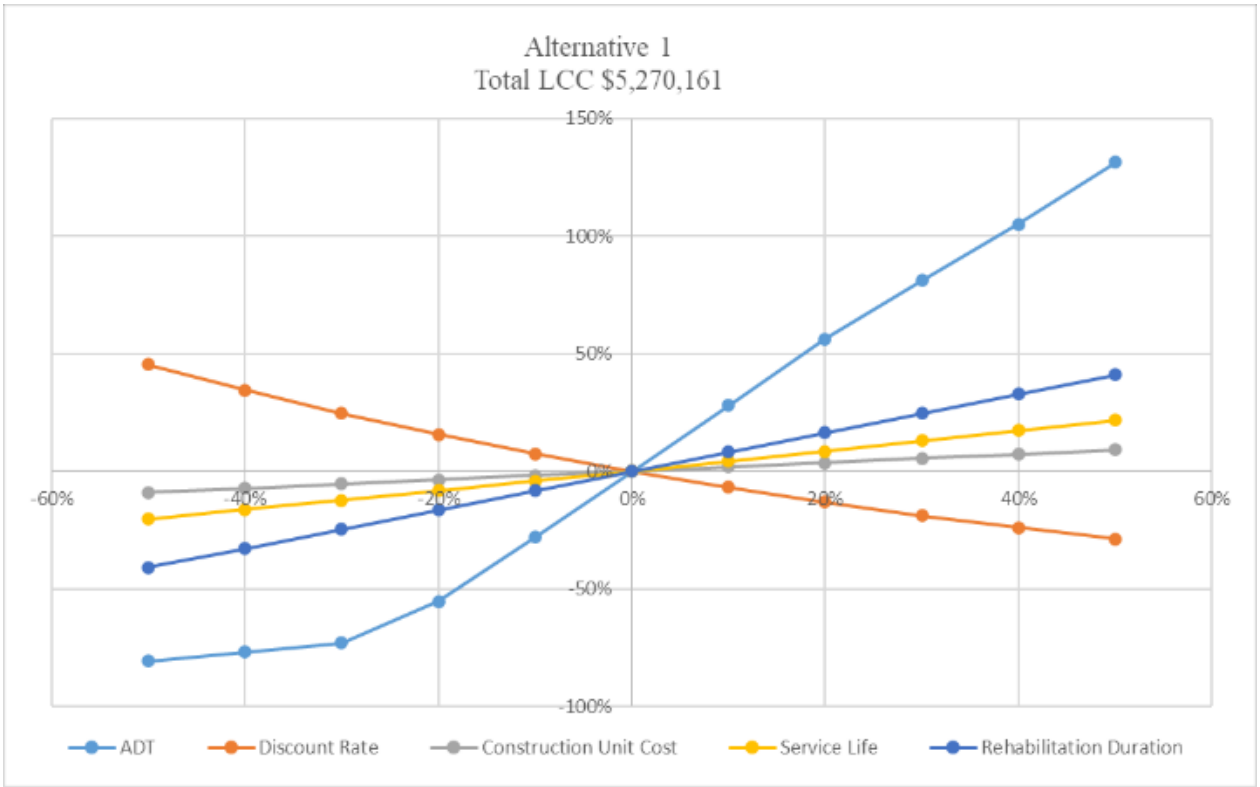
Alternative 2 with UHPC resulted in lower total life-cycle costs than alternative 1 with conventional concrete. Total agency costs were higher in alternative 2 with UHPC than total agency costs in alternative 1 with conventional concrete. However, total user costs are much higher in alternative 1 when compared to the total user costs in alternative 2. The user cost versus agency cost ratio in alternative 1 is also higher than the user cost versus agency cost ratio in alternative 2 (4.54 versus 0.78). At the end of the analysis, when agency and user costs were considered in the LCCA, the total life-cost in alternative 1 was about three times higher than the total life-cycle cost in alternative 2.

The sensitivity analysis of the variables involved in the LCCA revealed that the Average Daily Traffic (ADT) and construction duration of the rehabilitation activity have the most significant effects on the total life cycle cost for alternative 1.

Figure 49 shows the results of the sensitivity analysis in spider plots for alternatives 1 and 2 with the influence of the variables in LCCA results. The horizontal axis shows the change of the variable values from positive 50% to negative 50%. The vertical axis shows the change of total life cycle costs as the values of each variable changes.

In alternative 1, the spider plot unfolds that if the ADT increases in 50%, then the total life cycle cost increases in about 150%, and if the duration of the rehabilitation activity increases by 50%, the total life cycle cost increases approximately 50%. It is also observed that the sensitivity of the total life cycle costs to the ADT and duration of the rehabilitation activity in alternative 1 is higher than in alternative 2. The reason is that the construction time in alternative 2 with UHPC and ABC techniques is significantly less than alternative 1 with conventional concrete (4 days versus 14 days).

In alternative 2, the total life cycle cost is more sensitive to the construction unit cost when compared to alternative 1. The line of the construction unit cost in the spider plot overlaps with the duration of the rehabilitation activity in alternative 2, and both variables have similar effects in the total life-cycle cost under this alternative. Overall, agency costs in alternative 2 have more influence in the total life cycle cost when compared to alternative 1.



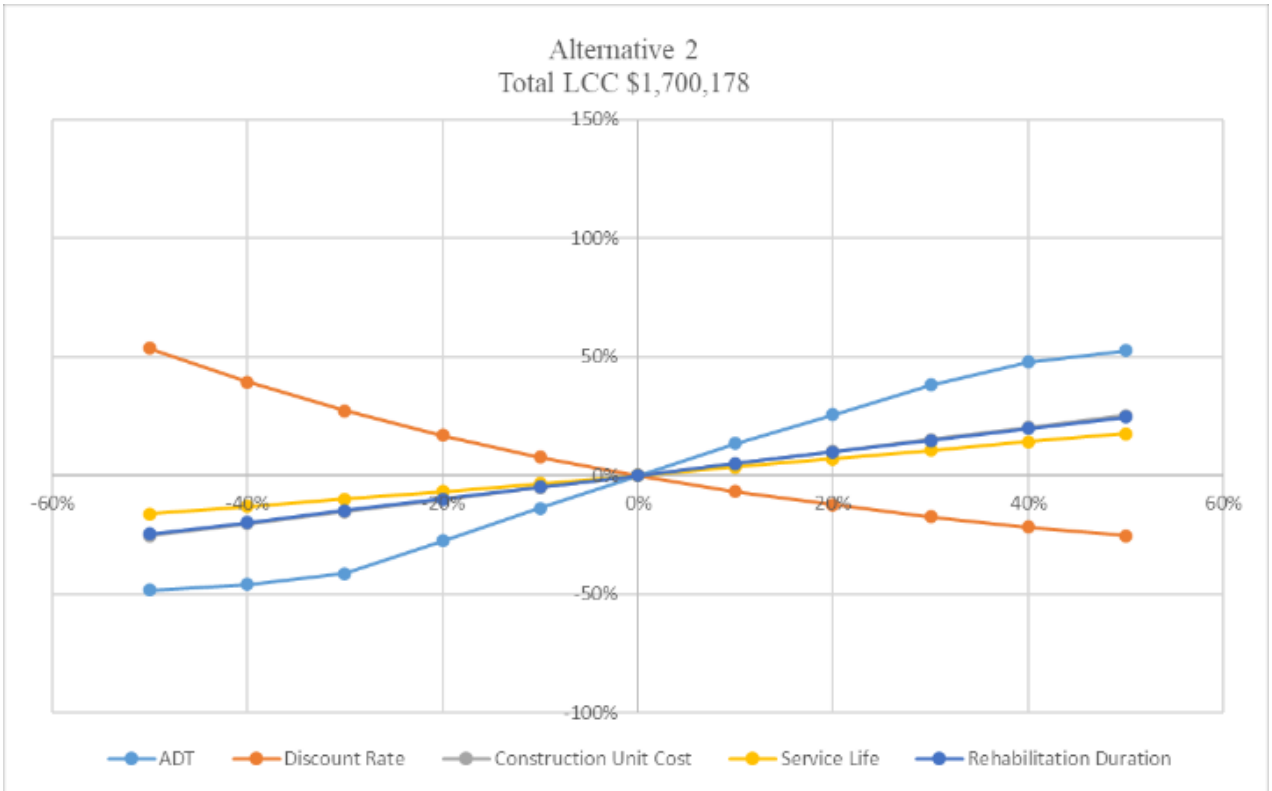


Figure 49. Sensitivity analysis spider plots for CSC and UHPC retrofitting alternatives.

6.4 Recommendations for Future Research

a. A performance-based criterion approach that relies upon material properties and expected behavior is recommended to evaluate the cost-effectiveness of their usage in retrofitting techniques. One major potential benefit of UHPC is its application to connect precast elements. In prefabricated elements, the usage of field-cast UHPC instead of traditional cementitious grout-like materials for closure pours has several technical advantages. A reason to use UHPC on bridge connections and girders is the increase in tensile strength. Because of the UHPC's mechanical properties, field-cast connections can be smaller and more durable. In addition, repairs with UHPC reduce the time of traffic closures.

b. Life expectancy reliable models are required to predict the performance and concrete condition over time. There are LCCA software tools with life-expectancy deterioration models for conventional concrete although not for UHPC. A major challenge to develop life-expectancy models is that there are limited records of UHPC performance. Performance models based on condition inspection records are desired.

   At present, National Bridge Index (NBI) condition ratings for conventional bridges are found on the Long-Term Bridge Performance (LTBP) database that can be access on-line through an open website. Condition inspections are conducted periodically to report the NBI with 9 being excellent condition and 0 absolute failure. A bridge is considered structurally poor if at least one of its components has a condition rating of 4 or less. Further research is required to model the different factors that influence UHPC life expectancy. A life-expectancy hybrid model that combines corrosion concrete deterioration equations and NBI to predict performance is a topic for future research. This model would have practical applications to identify cost-effective treatments and budgets needs to support management decisions at the network and project levels.

c. The performance-based framework developed in this research to select retrofitting materials can be used to evaluate new UHPC mix designs. UHPC has a wide variety of proprietary and non-proprietary mix designs according to the application. The type of concrete mix has a substantial impact on life-cycle costs due to their technical characteristics, construction process, and expected service life. At present, there is no consensus on a standard UHPC mix design and it varies with the project requirements. The development and implementation of a database with UHPC mix design properties and costs is aimed to support LCCA.

d. The development of a software tool to implement the step-by-step LCCA methodology developed in this research could facilitate the calculations to evaluate more alternatives for UHPC applications in ABC projects. The LCCA software tool could be connected to a UHPC mix design database that includes initial construction, maintenance, and rehabilitation costs. The database should also include information regarding user costs associated to the life-cycle activities. The user cost main components include traffic demand, traffic growth factors, frequency of maintenance activities, and lane closure times. It also crucial to define the work zone schedule to determine the user cost components finally used in the LCCA.

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