Hossain and Chang
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ORIGINAL INNOVATION

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Modeling life expectancy and cost effectiveness for UHPC bridge retrofitting techniques

Abid Hossain[1*] and Carlos M. Chang[1]

*Correspondence: ahoss019@fiu.edu

[1] Department of Civil and Environmental Engineering, Florida International University, Miami, FL 33174, USA

Abstract

Bridge components are subject to both structural loads and environmental stressors, rendering them susceptible to accelerated deterioration and potential collapse in the absence of effective maintenance and rehabilitation strategies. Moreover, the phenomenon of wet-dry cycling, coupled with elevated chloride concentrations prevalent in coastal regions, further expedites the degradation process of bridges, thereby escalating maintenance frequency and repair costs. In response to this challenge, the integration of innovative materials such as Ultra High-Performance Concrete (UHPC) is being explored for the development and implementation of maintenance and rehabilitation strategies. This study presents a comparative analysis between conventional methods and UHPC applications for bridge repairs, utilizing Life Cycle Cost Analysis (LCCA) to encompass both agency and user costs, and applies Monte Carlo simulation to account for the variability of the modeling factors. A practical case study illustrates the applicability of the LCCA methodology, revealing that the utilization of UHPC contributes to a reduction in the total life cycle cost for bridge maintenance and rehabilitation. Life expectancy, Average Daily Traffic (ADT), and the duration of construction activities during rehabilitation emerge as the most influential factors affecting life cycle costs. The main contributions of the study are the development of the life-expectancy model and step-by-step Life-Cycle Cost Analysis (LCCA) methodology. Findings from this study aim to identify cost-effective retrofitting techniques for maintaining bridges in a "State of Good Repair."

Keywords: Ultra-high-performance concrete (UHPC), Cost-effectiveness, Life cycle cost analysis, Monte Carlo simulation

1 Introduction

Concrete technology has greatly progressed during the last two decades. To cope with modern construction techniques, concrete has gone through rigorous research regarding mix design, mechanical properties, and durability. A result of the extensive effort is Ultra-High-Performance Concrete (UHPC), an innovative material with the potential to become a viable alternative for improving the sustainability of bridge components. While reliable in many scenarios, traditional, normal-strength concrete presents certain challenges, especially in the realms of rapid construction and bridge element restoration.

Larrad initially conceptualized UHPC (de Larrard and Sedran 1994). UHPC is an exceptional cementitious material, durable against the freeze-thaw attack and permeation of gases and liquids. It has a low water-to-cement ratio (less than 0.25) and a low maximum grain size diameter with the addition of pozzolanic filler materials like silica fume (American Concrete Institute (ACI) 2018). Due to its versatile applicability, UHPC can be used in preventive maintenance and rehabilitation activities. UHPC can be utilized in overlays, closure joints, claddings, and shells to preserve or rehabilitate bridge concrete decks, girders, or columns (Graybeal 2011). The entire deck, beam, and bridge systems can be built with UHPC to replace damaged elements.

The motivation for this research stems from the versatile applicability of UHPC in bridge element repair. UHPC has proven its capability as a repair material for corroded steel bridge girders (McMullen and Zaghi 2019). To recognize the potential benefits of UHPC applications, conventional retrofitting techniques should be compared to UHPC applications. Life-cycle cost analysis (LCCA) is a tool that can assist in comparing treatment solutions for bridge maintenance strategies. UHPC is a comparatively recent invention in terms of concrete materials. The challenges for the wide use of these materials are generally due to the need for more quantitative tools to evaluate the structural performance over time. This research study aims to apply life cycle cost analysis to compare UHPC applications with conventional bridge repair techniques. A major part of the life cycle cost analysis is to define a deterioration model for UHPC applications. A model is proposed in this research to accommodate the life expectancy of UHPC applications in the life cycle cost methodology.

1.1 Applicability of UHPC as a repair and rehabilitation material

UHPC is a material used in both preventive maintenance and rehabilitation bridge activities. The most common bridge activities are deck closure pours for precast deck elements, bridge deck overlay, and shell encapsulation. 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 a combination of axially and laterally loaded substructural reinforced concrete (e.g., bridge columns), replacing 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). 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 is effectively used as a shear connector in longitudinal and transverse connecting joints (Russel and Graybeal 2013). UHPC overlays are a preventive measure in conjunction with spot repairs of isolated distresses (Shann 2012). UHPC has a low water-cement ratio, high binder content, and optimum packing density that eliminates capillary pores by providing a dense matrix (Naaman and Wille 2012). Studies have developed an innovative link slab design utilizing UHPC to eliminate transverse deck joints wherever feasible (Royce 2016). For bridge component replacement, UHPC can be used on an I girder, box girder, deck slab, arch ring, or even whole superstructure (Graybeal et al. 2020). UHPC is used as a combined application of UHPC deck with steel box girder to achieve a lighter composite girder bridge. UHPC deck slab improves the fatigue performance of the slab (Brühwiler 2018). Full-depth waffle deck panels with UHPC, hollow-core slabs with UHPC faces, and composite slabs made by High-Performance Concrete (HPC) and UHPC demonstrated the material's characteristics to be both lightweight and strong (Wang et al. 2021). 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). UHPC Box-girders, U-shaped girders with precast slabs, reversed U-shaped girders, and even Pi-shaped arch bridges are now constructed with greater efficiency and enhanced performance characteristics (Abdal et al. 2023).

Cost savings are achieved using partial prestressing in UHPC pi-girder design (Federal Highway Administration (FHWA) 2014). The high strength of UHPC results in a substantial reduction of dead-load and less restricted structural member shapes (Plevny 2020). UHPC allows innovative bridge element replacement techniques that accelerate the rehabilitation process, extending the bridge service life with minimum road user delays and community disruptions (Aaleti et al. 2013). Compared to conventional concrete, UHPC allows longer-span bridge structures with smaller member sizes and 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). Using 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 using high-performance materials to prefabricate bridge elements. High-performance concrete and large diameter prestressing strands in fabricating precast/prestressed I-girders displayed superior shear and flexure capacity, resulting in smaller girder sections, higher span-to-depth ratio, and increased girder's centerline spacings (Haber et al. 2018).

The paper is organized into three sections. The introduction outlines the capability and versatility of UHPC as a repair material for bridge structures. The Section 2 is employed for evaluating UHPC applications, drawing on a life cycle cost analysis approach. This method, adapted from a Federal Highway Administration study, is crucial for estimating the lifespan of UHPC compared to conventional concrete (Walls and Smith 1998). A pivotal part of the study is developing a life expectancy model for UHPC. Existing models were reviewed, and a novel approach tailored to UHPC was introduced, emphasizing its enhanced durability and potential for longer service life. The model considers various factors, such as chloride-induced corrosion, to predict the performance and longevity of bridge components. The Section 3 is a practical demonstration of the life cycle cost analysis and the proposed life expectancy model. It assesses the comparative performance and cost-effectiveness of UHPC versus conventional concrete in a real-world scenario, providing tangible insights into the benefits of UHPC in bridge maintenance.

2 Methodology

To evaluate the cost-effectiveness of Ultra-High-Performance Concrete (UHPC) applications, this study employs a life cycle cost analysis, a widely recognized method. The study carried out a deterministic life cycle cost analysis. The life cycle cost methodology is derived from a Federal Highway Administration (FHWA) study previously applied on pavements (Walls and Smith 1998). Subsequent sections will delve into the details of the life cycle cost approach and the process of determining life expectancy.

2.1 Life cycle cost

Life Cycle Cost Analysis (LCCA) is an economic project assessment 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 life cycle cost over the expected service period. 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.

A deterministic life cycle cost analysis model was considered for the analysis. The methodology was proposed by Walls and Smith (1998) under a FHWA-sponsored research project (Walls and Smith 1998). The methodology considers the Net Present Value as the economic efficiency indicator. Life cycle costs over a specified period are discounted to the present to calculate the Net Present Value (NPV) using the following equation:

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

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 the analysis period

n2 = Number of rehabilitation activities over the analysis period

N = Length of analysis Period

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

$$EUAC = LCC(NPV)\frac{r.(1+r)^{N}}{(1+r)^{N}-1} \tag{2}$$

Where:

N = Length of the analysis period

r = Discount rate

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 may be adequate for determining when a deck overlay should be scheduled for a standard highway design (e.g., ten years). 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 (American Association of State Highway Transportation Officials (AASHTO) 2009). The life expectancy of UHPC is greater than conventional concrete, and it can reach up to 100 years (Farzad et al. 2020). Based on the life expectancy of new concrete structural materials, the analysis period might be even extended over 100 years (Chang et al. 2016). Previous studies have used 75 years to compare bridge systems with UHPC and conventional concrete (Dong 2018). For this research study, a 60-year analysis period was considered to estimate the life cycle costs of the alternatives. The 60-year analysis period is based on expert knowledge, and it aims to cover life expectancy for most bridges, in particular, for conventional concrete. The life cycle cost analysis includes a salvage value component when the model predicts life spans longer than 60 years for UHPC.

2.1.1 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 the length of the analysis period. Initial construction, maintenance, and rehabilitation costs include material, labor, and equipment.

2.1.2 User costs

User cost estimates are based on time delays, vehicle operation, and crashes (Federal Highway Administration (FHWA) 2002; Watts et al. 2012). 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. A twelve-step procedure for calculating user costs was followed based on the FHWA study (Walls and Smith 1998).

2.1.3 Sensitivity analysis

Sensitivity analysis is used to identify input parameters that could impact the total life-cycle cost most. Sensitivity analysis is 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 could be used to represent the sensitivity analysis results. 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 (National Cooperative Research program (NCHRP) 2012).

2.2 Development of life expectancy model

To determine the best cost-effective maintenance strategy, it is important to understand the deterioration mechanism of bridge components. The deterioration process is modeled for reinforced concrete elements as a function of the steel corrosion affecting reinforced concrete bridge elements.

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. The existing life expectancy models and techniques were thoroughly reviewed. The deterministic models include chloride chloride-induced corrosion model and the carbonation-induced corrosion model. The probabilistic models can be state-based, or time-based. The techniques for the models can be through regression, reliability-based models (e.g., Monte Carlo Simulation), or Artificial Intelligence-based. Some studies employed a comprehensive multi-physics simulation approach (Fan et al. 2022). Mechanical responses can be integrated into multi-physics models and include factors like cracking effects on mass transport, electrochemical processes, and mechanical responses to model steel corrosion in concrete. Time-dependent multi-physics modeling considers factors like chloride diffusion, mechanical loading, and regional environmental conditions in life expectancy modeling (Fan et al. 2023). One study simulated micro- and macro-cell corrosion processes, electrical potential distribution, and oxygen concentration within the concrete cover (Cao et al. 2013). It utilizes a thick-walled cylinder model for mechanical analysis of cover cracking. The outcomes highlight that macrocell corrosion rates remain relatively stable, while microcell corrosion rates increase significantly as oxygen permeability increases due to corrosion-induced cover cracks. The model provides insights into the dynamic corrosion rates in RC structures, considering the impact of cover cracking on the corrosion process. The multi-physics model incorporates various physical, chemical, and electrochemical processes, including the thermodynamics and kinetics of corrosion, the impact of environmental factors like temperature, humidity, and oxygen, and the expansive nature of corrosion products (Lepech et al. 2016). Another study applied finite element modeling on OpenSees considering parameters like intervention area length, machined index, and repair timing (Pelle et al. 2023). The main outcomes reveal the efficacy of UHPFRC in enhancing structural performance, the importance of timely repair, and offer design recommendations for effective repair strategies. Highly ductile concrete materials exhibit restrained crack widths, enhanced freeze-thaw, and salt-scaling performance, and improved resistance to chloride penetration and corrosion propagation (Bandelt et al. 2023). These studies show that UHPC slow the structural deterioration in bridge decks when compared to normal-strength concrete systems.

At present, there is no integrated model to predict life expectancy for UHPC applications. LCCA software like Life 365 is based on this life expectancy model but does not include UHPC. The most common life expectancy model for bridge structures is determined from the FHWA bridge condition rating. UHPC being a comparatively recent concrete material, there needs to be more bridge condition rating data for UHPC bridge applications to build a data-driven model. In this research study, the life expectancy of UHPC and conventional concrete has been modeled based on the deterioration due to chloride ingress into the concrete.

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. Concerning chloride-induced corrosion, the thickness and permeability of the concrete cover determine the structure's resistance to chloride ingress. Most often the quality of the concrete is expressed in terms of a diffusion coefficient. The most common equations to describe chloride ingress into concrete are based on Fick's second law of diffusion called the error function solution. Crank's closed-form solution of Fick's second law of diffusion for a semi-infinite medium is (Morcous and Lounis 2007):

$$C(x,t) = Cs[1 - \mathrm{erf}(\frac{x}{2\sqrt{Dt}})] \tag{3}$$

Where:
Cs = Surface chloride concentration
D = Diffusion coefficient of chlorides
x = Cover depth
t = Time to corrosion initiation
The corrosion is initiated in two phases:

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 period from the initial exposure to chlorides until the onset of corrosion.

Propagation Stage

Post-corrosion stage corresponds to the initiation of damage (e.g., cracking, delamination, spall). 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 are obtained from core samples extracted from the deteriorated bridge. Then, both parameters are derived by fitting Fick's second law of diffusion to the observed chloride profile.

An acceptance criterion is established to model the deterioration due to chloride diffusion. Concerning chloride ingress, it is generally agreed that for new concrete structures, the end of service life is reached when the embedded steel reinforcement is de-passivated. This implies that service life is defined by when the reinforced steel reaches the critical chloride content. A Monte Carlo simulation-based model is used 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 inversely solving Fick's second law, the time to initiation can be determined from the following equation:

$$t = \frac{C \times erf^{-1}(1 - \frac{C}{Cs})^{-2}}{4D} \tag{4}$$

Where:

t = Time to corrosion initiation Cs = 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 are:

- The steel is initially protected from corrosion by the chloride-free concrete surrounding 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 of 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, most mathematical, and empirical functions show a linear relationship between rebar loss section and crack width propagation.

2.2.1 *Defining 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, and 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 log-normal distribution (Morcous and Lounis 2007). Tables 1 and 2 below shows the variables and the values adopted for the input for conventional strength concrete and UHPC:

Table 1 Conventional strength concrete properties for life expectancy model

Variable	Unit	Mean value	Standard Deviation	Distribution Type	Reference
Surface Chloride Concentration	kg/m3	5.47	1.83	Lognormal	(Morcous and Lounis 2007; Farzad 2018)
Diffusion Coefficient	mm2/year	31.536	9.2	normal	Farzad 2018
Water Cement Ratio	n/a	0.5	n/a	Fixed value	Federal Highway Administration (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 2 UHPC properties for life expectancy model

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	Federal Highway Administration (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 ($1.3 \times 10-13$ m2 /s or 3.15 mm2/year) (Federal Highway Administration (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 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 such as the diffusion coefficient are debated as to whether they should be considered constant or time dependent. For this study, the diffusion coefficient is assumed to be constant.

2.2.2 Monte Carlo simulation

This approach offers a dynamic and probabilistic assessment of spall occurrence over the structure's lifespan. The methodology begins by deconstructing the concrete section into smaller elements. These elements are assumed to have similar properties although they differ in their probabilities of manifesting corrosion. This division into smaller sections is crucial for a detailed analysis, as it allows for assessing localized variations within the bridge structure. Incorporating the Monte Carlo simulation approach, the methodology assigns separate probabilities for corrosion to each of these small sections. This probabilistic model captures the inherent randomness and uncertainty in the variables. Truncation factors are applied in the simulation, representing the input parameters' lower and upper bounds to ensure they are within realistic and physically meaningful ranges. A random number of 220,000 samples were used in the simulation. This means the simulation runs 220,000 iterations, each representing a unique set of conditions or parameters, to statistically analyze the corrosion process and its impact on the concrete structure. This large number of samples allows for a comprehensive and statistically robust analysis, capturing a wide range of possible outcomes and providing a detailed understanding of the corrosion behavior over time.

2.2.3 Consideration of crack diffusivity

A critical aspect of this approach is the consideration of crack diffusivity. The presence of cracks in the concrete significantly alters the diffusivity of chlorides, accelerating the corrosion process. The methodology uses derating factors to model the increased diffusivity in cracked elements, a technique inspired by the research of Sagüés et al. Crack diffusivity on the concrete element is modeled with derating factors that assume a crack percentage in the concrete when constructed (Sagüés et al. 2014). The derating factor is a multiplier that modifies the crack diffusivity value for some bridge elements. Each element in the derating factor array corresponds to a specific condition or scenario within the bridge structure. This research highlighted the importance of monitoring chloride levels near cracks and at reinforcing surfaces, as these are areas where local transport conditions can significantly influence the evolution of damage.

Crack diffusivity in concrete is addressed through a probabilistic approach. A crack diffusivity ratio and its standard deviation indicate how much more permeable cracked elements are compared to uncracked. The simulation then generates a random value for each element in the simulation, determining whether it has increased diffusivity due to cracks. Sampling from from a normal distribution and it is checked if the sampled value, adjusted by the standard deviation, falls within predefined bounds. The following variables are considered in the simulation:

> Crack Fraction: This represents the proportion of elements in the concrete section that contain cracks. It influences how widely crack-enhanced diffusivity affects the structure.
>
> Crack Diffusivity Ratio: This factor quantifies how much more permeable cracked elements are compared to uncracked elements. A higher ratio indicates a greater increase in diffusivity due to cracking.
>
> Standard Deviation of Crack Diffusivity: This parameter accounts for the variability in how cracks affect diffusivity across the elements in the concrete section. It's used in the probabilistic model to generate a range of diffusivity values for cracked elements.

These factors are combined in a Monte Carlo simulation to assess the impact of cracking on the overall diffusivity of the concrete structure, which is crucial for understanding chloride ingress and subsequent corrosion risks in reinforced concrete.

2.2.4 Percentage of spall calculation

By simulating numerous scenarios, the Monte Carlo simulation provides a statistical distribution of possible outcomes for each element's degradation over time. The cumulative damage to the structure is then assessed by considering the combined effect of all individual elements. Unlike a straightforward multiplication of probabilities, this method accounts for the interdependencies and collective impact of the elements' degradation processes. This is particularly important in understanding how localized damage, like spalling, can affect the structure's overall integrity. The time to corrosion for each simulated element, factoring in the variables is calculated. Following this, the simulation process tracks the corrosion progress over time, marking each year with the tally of elements that have reached the corrosion threshold. This tracking process is enhanced by the simulation's ability to handle a large number of elements by considering a large number of samples (220,000) in the Monte Carlo simulation, allowing for a granular and accurate representation of the corrosion process. The spall frequency is then calculated by dividing the corroded elements or sectors in the concrete section by the total, which is normalized over the simulation's timeframe to yield a cumulative spall percentage. These results in a detailed time-based profile of spall occurrence, reflecting corrosion's nuanced and progressive nature and its impact on bridge elements. The output of the simulation provides valuable insights into the expected lifespan and maintenance requirements of different types of concrete. For instance, it can compare the performance of conventional concrete against more advanced materials like Ultra-High-Performance Concrete (UHPC) under similar conditions. The simulation results help in predicting the time frame in which significant spalling damage might occur, guiding maintenance and repair planning. From the simulation results. Figure 1 shows that more than 40% of spalling damage is expected in 30 years for conventional concrete, while UHPC is expected to have 40% of spalling damage in 80 years.

3 Case study

In this case study, two structural entities: one utilizing conventional concrete and the other incorporating Ultra-High-Performance Concrete (UHPC) are evaluated. Given UHPC's common applications, as previously summarized, the case study focuses on two key uses: deck surface overlays and closure joints, both integral to the analysis. The specific bridge deck area under consideration spans 1,375 square feet, featuring a slab width of 42 feet. Within this context, the case study evaluates two distinct retrofitting alternatives for construction:

> *Alternative 1:* Alternative 1 involves a conventional cast-in-place concrete deck slab. This approach represents the traditional method, emphasizing the use of standard concrete materials and construction techniques.
>
> *Alternative 2:* Alternative 2 proposes a precast deck slab, uniquely integrated with UHPC for the closure joints. This option explores the advantages of precast components, combined with the enhanced properties of UHPC, potentially offering improved durability and longevity.

3.1 Alternative 1

To evaluate the alternatives, LCCA is conducted over 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. Type II concrete is considered conventional concrete in this case study. 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

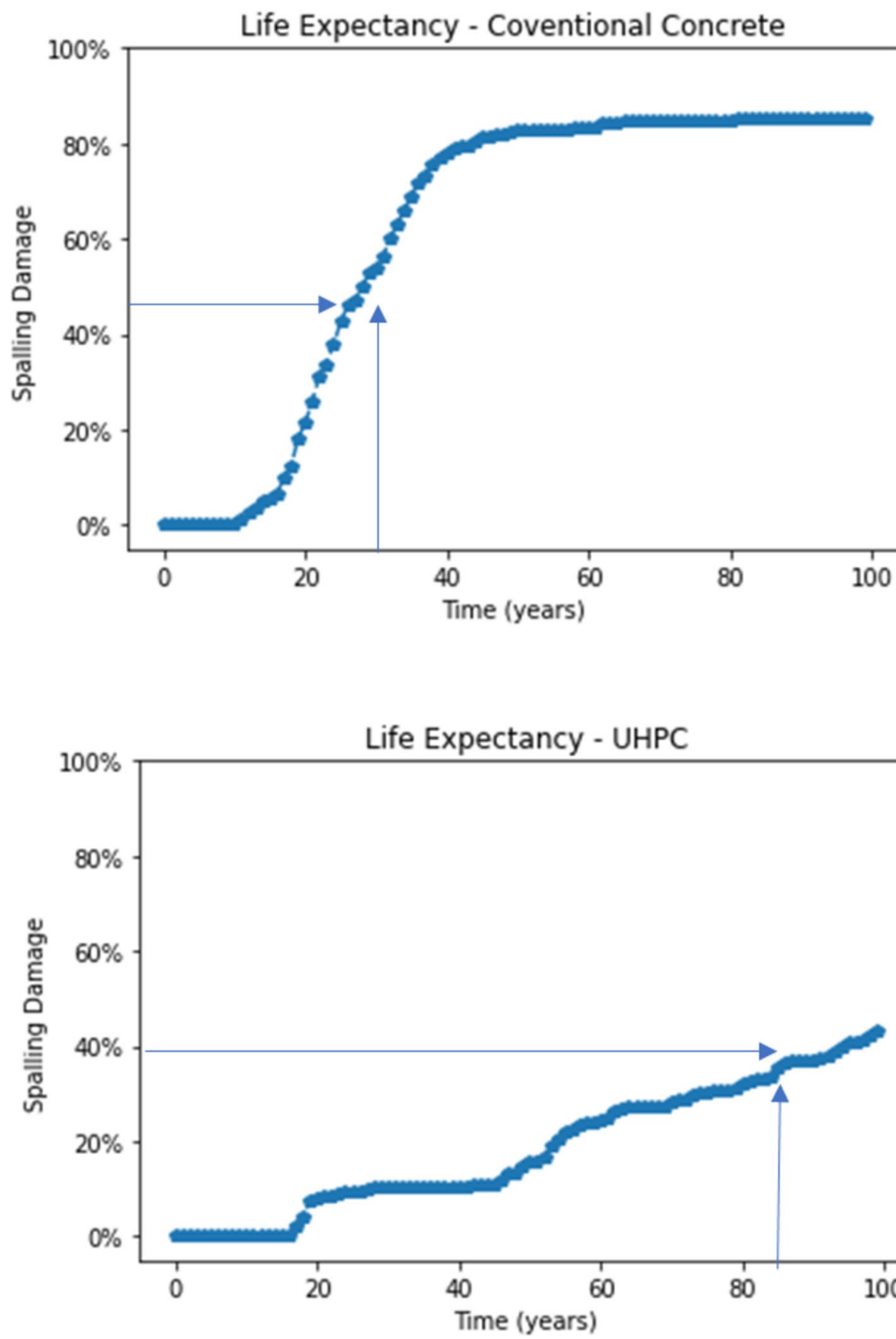

Fig. 1 Life expectancy diagram of conventional concrete and UHPC

is less than 500 ppm, and the sulfate content is between 150 and 1000 ppm (Florida Department of Transportation (FDOT) 2022). Table 3 shows the properties for the type II concrete considered for Alternative 1.

The life expectancy of the deck slab is calculated based on the model described above. 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

Table 3 Properties of type II concrete for alternative 1

Properties	Typical Values
Air content	1 to 6%
Specified Minimum Strength (28 days)	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: Florida Department of Transportation (FDOT) 2022

Table 4 Initial construction cost estimation of 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	274,600	$288,330
Expansion Joint	Per Linear ft	45	42.25	$1901
	Total			$633,981

calculated and spalling concrete slab damage is projected over time. For cast-in-place (CIP) conventional concrete slabs, the spalling damage evolution over time is observed in Fig. 1. In this case study, it is considered that the end of the service life 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.

In terms of maintenance activity, it is considered that a thin bonded epoxy overlay is scheduled every 10 years as preventive maintenance (Chang et al. 2016). The conventional concrete slab has a life expectancy of 30 years. Therefore, rehabilitation with the replacement of the concrete slab is scheduled for year 30. A new life cycle for the conventional concrete slab begins at year 60. At year 60, there is no remaining life nor a salvage value.

Table 4 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 (Florida Department of Transportation (FDOT) 2020).

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 5 shows the agency costs throughout the analysis with calculations of the present cost using a 3% discount factor.

3.1.1 User cost

User cost calculations follow the FHWA method (Walls and Smith 1998). In this method, user cost estimates are based on traffic projections distributed by periods during the day. The user cost is calculated for the rehabilitation activity in year 30. The 24-hour work schedule for the rehabilitation activity is constructed with distributed work and non-work zone periods. The projected Annual Average Daily Traffic (AADT) is assumed to be 114,000

Table 5 Agency cost 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

It is assumed in the analysis that there is no inflation

at year 30. From the FHWA research study, the user cost 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 of Traffic Affected by Each Cost Component.
Step 6 Compute Reduced Speed Delay.
Step 7 Select and Assign Vehicle Operating Cost (VOC) rates.
Step 8 Select and Assign Delay Cost Rates.
Step 9 Assign traffic to vehicle classes.
Step 10 Compute individual user cost components by vehicle class.
Step 11 Total work zone user costs.
Step 12 Address circuitry and crash costs.

Crash costs were not included in the analysis. The calculation was simulated by constructing an Excel file following the methodology. The detailed calculations of the user cost can be found in the report by (Chang and Hossain 2019).

$$\text{Total User Costs at year 30} = \text{Total VOC for inbound vehicles} + \text{Total Delay Cost}$$

$$\text{Total User Costs at year 30} = \$613,044 + \$9,871 = \$10,484,364$$

$$\text{Present User Cost at year 0} = (1/\ (1+0.03)\hat{}30) \times 1,048,364 = \$4,319,557$$

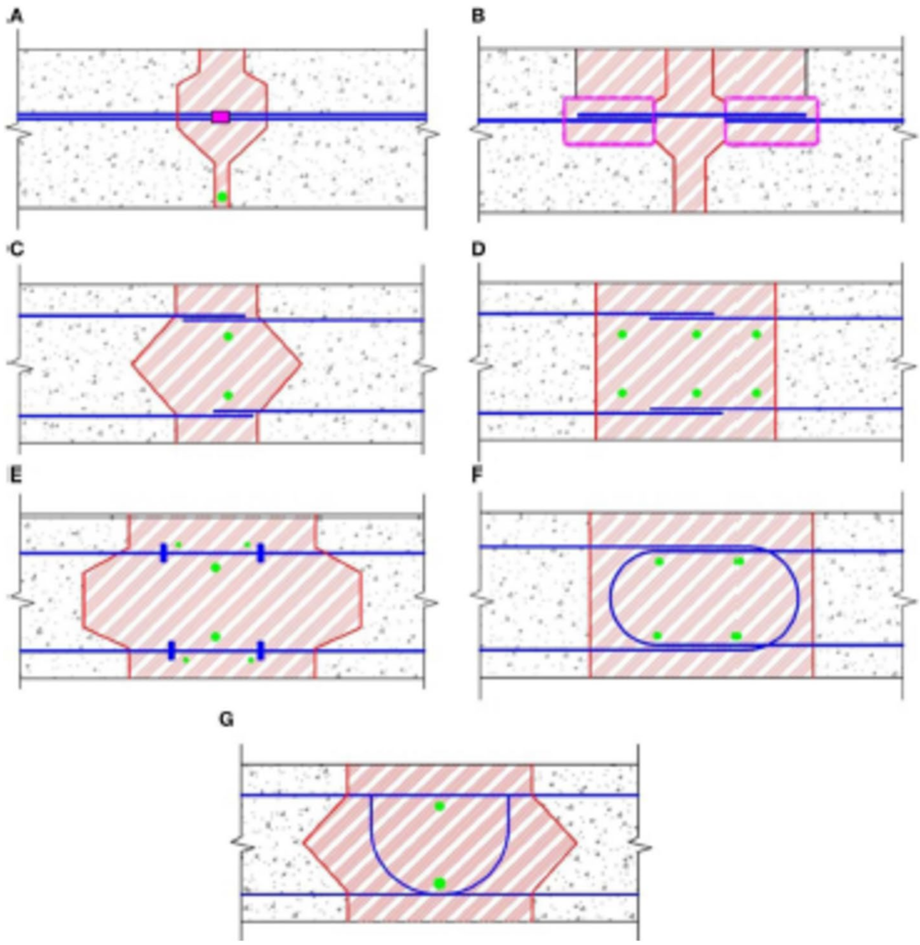

Fig. 2 Schematic configurations for common closure joints. **A** post-tensioning, (**B**) mechanical connectors, (**C**) ultra-high performance with straight bars, (**D**) normal-strength concrete with straight bars, (**E**) normal-strength concrete with headed bars, (**F**) normal-strength concrete with 180○ hooked bar, (**G**) normal-strength concrete with 90○ hooked bar

3.2 Alternative 2

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 Accelerated Bridge Construction (ABC) techniques are shown in Fig. 2 (Jaberi Jahromi et al. 2020). The joint configuration for the project corresponds to Fig. 2C. The prefabricated bridge deck is also made with UHPC. Construction can be completed in 4 days at the site according to FDOT information.

The UHPC mix design reference is from a research study conducted in 2021 for non-proprietary UHPC transverse field joints (Abokifa and Moustafa 2021). The mix design and main mechanical properties of the UHPC mix are presented in Table 6.

The difference with the deterioration model used for alternative 1 is in the parameters of the equations that change according to 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 over time. It is observed from Fig. 1 that 40% of deck spalling damage of the slab is expected to

Table 6 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 and Moustafa 2021

be reached in 80 years. There is no full slab deck replacement in this alternative since the analysis period for the LCCA 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.

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.

3.2.1 Agency costs

The initial construction cost was from an FDOT project that consisted 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. For this project, the UHPC unit cost for an overlay is $375 per cubic foot. Therefore, the unit cost for the 1-inch UHPC overlay is: $375 $\times$ 0.083 = 31 $/sq-feet (Note: 1 inch = 0.083 ft).

For preventive maintenance, it is assumed that 10% of the deck needs overlay repairs every 15 years. Therefore, the cost per preventive maintenance activity is: 31 $\times$ 0.10 $\times$ 137.5 sq-feet = $4,263. At year 60, the bridge slab deck has 20 years of remaining life and consequently a salvage value. There has yet to be a 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 concerning the cost of rehabilitation. In this case study, the remaining life of the pre-cast slab is 25% of the total

Table 7 Agency 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

Table 8 Summary of life cycle costs

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

expected service life of 80 years. Therefore, the salvage value is estimated at 25% of the initial construction cost.

$$\text{Salvage value at year 60}: \ 980,000 \times 25\% = \$245,000$$

Table 7 shows the agency costs throughout the analysis with calculations of the present cost using a discount rate of 3%.

User cost 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 in 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:

$$\text{Total VOC for alternative } 2 = \$175,155$$

$$\text{Total Delay Costs for Alternative } 2 = 2,820,377$$

$$\text{Total Present User Costs} = (\$175,155 + \$2,820,377) \times (1/\ (1+0.03)^{47}) = \$746,662$$

An overview of the LCCA results for the two alternatives is shown in Table 8.

4 Results and discussion

Agency cost in alternative 1 with conventional concrete is higher than alternative 2 with UHPC, although the total NPV of alternative 1 with conventional concrete is higher in the long term. The final total NPV and EUAC cost is lower in alternative 2 with UHPC considering the user costs. Agency cost is about the same for both alternatives. However, the different frequencies of maintenance activities and costs 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). Accelerated Bridge Construction (ABC) is very commonly aggregated with the incorporation of UHPC. 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 that of conventional cast-in-place concrete deck slack. and this difference is reflected in the LCCA results.

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 in spalling damage over time in concrete bridges.

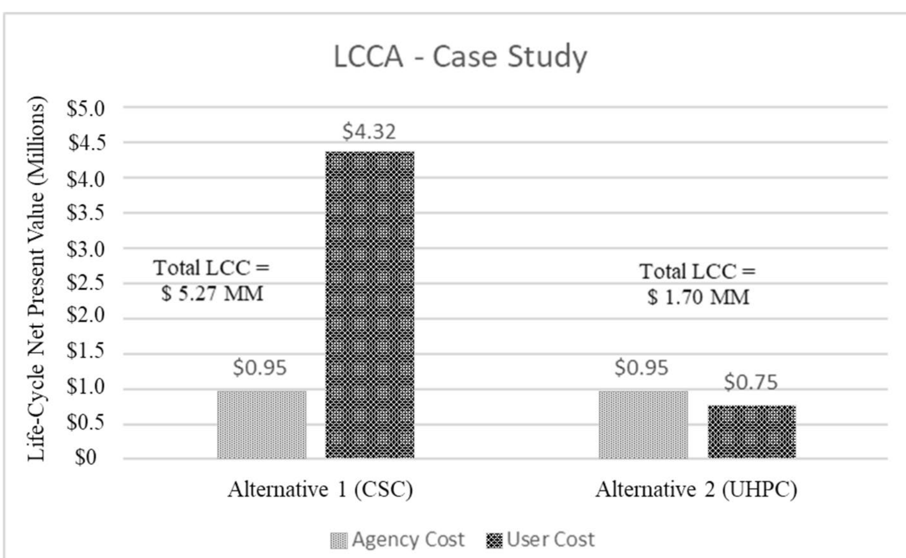

Fig. 3 Comparison of Net Present Value (NPV) life cycle costs for CSC and UHPC

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 3 shows the comparison of agency and user costs between the two alternatives under consideration in the case study.

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 Average Daily Traffic (ADT) has the most significant effect on the user's costs and as a result on the total life cycle cost. Other factors affecting the life cycle costs are the service life of the design and the duration of the rehabilitation activity. A sensitivity analysis was performed to evaluate the impact of various factors on the life cycle costs of two alternatives. This analysis involved varying individual variables – service life of concrete, construction unit cost, average daily traffic (ADT), rehabilitation duration, and discount rate by ±50%. The analysis revealed that for both alternatives, a 50% increase in service life significantly reduces life cycle costs, whereas a 50% increase in rehabilitation duration increases them. It was also found that ADT has the most significant effect on total life cycle costs for both alternatives. Furthermore, the total life cycle cost for alternative 2 is less sensitive to variations in ADT compared to alternative 1, which is attributed to the difference in construction days between the two alternatives (14 days versus 4 days). Additionally, the total life cycle cost of the alternative 2 is more sensitive to changes in construction unit cost.

5 Conclusion

The comparison of Ultra-High-Performance Concrete (UHPC) to conventional concrete for bridge decks concluded that UHPC's higher initial cost is balanced by its extended durability, prolonging the service life, and resulting in long-term savings. The results of the Monte Carlo simulation-based approach to estimate life expectancy including variables like surface chloride concentration, diffusion coefficient, and concrete cover depth. UHPC revealed a longer corrosion initiation phase due to its lower permeability and better resistance against chloride ingress. The life expectancy of UHPC shows potential for an 80-year service life without requiring full replacement while for conventional concrete is estimated at 30 years before major spalling damage occurs. These benefits, along with shorter construction times and lower user costs with Accelerated Bridge Construction methods, demonstrated UHPC's cost-effectiveness. Despite higher upfront costs, UHPC's reduced maintenance frequency, lower lifecycle costs, and extended service life make it a more economically viable option for bridge construction and repairs.

The research effort described in this manuscript serves as a foundational step, focusing on expanding the use of corrosion models with factors well-known to bridge practitioners. For this reason, the study's scope did not consider more novel scientific approaches for modeling, such as the consideration of electrochemical reactions, corrosion progression, cracking behavior, and time-dependent factors. Scientific corrosion models are documented in the literature review, and future research could integrate them into a holistic comprehensive approach. It is recognized that a holistic comprehensive approach should be able to provide additional insights about bridge structural durability and corrosive conditions, although the main conclusions of the study are expected to remain unchanged.

The main significant contributions of the study are the development of the life-expectancy model and step-by-step Life-Cycle Cost Analysis (LCCA) methodology. These contributions provide a strong basis and straightforward guidance for comparing bridge materials, both in construction and repair treatments. It underscores the vital need to cosider both upfront expenses and long- term costs when making decisions about infrastructure.

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Authors' contributions
AH and CC conceptualized the research. The original draft was written by AH, proofread, and edited by CC.

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Availability of data and materials
The datasets and codes used and/or analyzed during the current study are available from the corresponding author upon reasonable request.

Declarations

Competing interests
The authors declare that they have no competing interests.

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