

Article

Durability, Service Life, and Techno-Economic Analysis of Non-Proprietary UHPC

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Abstract: Non-proprietary ultra-high-performance concrete (UHPC) has been developed by numerous research institutes, each exhibiting unique characteristics distinct from both commercial and other non-proprietary UHPC formulations. However, comprehensive techno-economic analyses (TEA) comparing UHPC to conventional concrete, alongside extensive long-term durability assessments, remain limited. This study bridges these gaps by evaluating the long-term durability, modeling service life, and conducting a TEA of resource-efficient non-proprietary UHPCs. Durability characterization was conducted in accordance with ASTM standards, focusing on chloride ion penetration (via electrical surface resistivity), drying shrinkage, freeze–thaw (F–T) resistance, and absorption. These properties were monitored over a year to capture both transient and steady-state performance. A durability assessment factor (κ) was introduced to assess and compare the new UHPCs with existing alternatives. Results demonstrated that the UHPCs achieved high electrical surface resistivity, exceeding the low chloride ion penetration threshold of $21 \text{ k}\Omega\cdot\text{cm}$ within 1 week of casting. No deterioration was observed after 600 F–T cycles, indicating superior F–T resistance. Drying shrinkage remained below 0.1%, and absorption remained below 1.4%. These results highlight the long-term durability of the UHPCs, with a projected design service life surpassing 350 years, aligning closely with proprietary UHPC products.

Keywords: absorption; drying shrinkage; durability; electrical surface resistivity; freeze–thaw (F–T); techno-economic analysis (TEA); ultra-high-performance concrete (UHPC)



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1. Introduction

In the United States, infrastructures such as bridges, roads, and buildings predominantly consist of concrete structures. The American Society of Civil Engineers (ASCE) frequently rates these infrastructures as poor, assigning a grade of D+ until 2020 and slightly improving it to a C- in 2021, primarily due to durability and aging issues [1,2]. According to the 2023 data from the Federal Highway Administration (FHWA) National Bridge Inventory (NBI), a significant portion of bridges, approximately 42%, are over 50 years old, with about 36% requiring repair or replacement. Additionally, about 6.8% of bridges, totaling around 46,000, are structurally deficient, although a gradual annual improvement trend is observed. The estimated cost to address the necessary repairs and replacements of these bridges is staggering around USD 319 billion.

Durability issues in concrete structures stem from various weathering actions such as freeze–thaw (F–T), chemical attacks from sulfates and chlorides, winter salts and brines, high temperatures, low humidity, and abrasion. These durability problems, along with increasing structural loading demands, degrade concrete structures, leading to higher

maintenance and repair costs. Additionally, increasing concerns about climate change demand that structures withstand extreme weather events such as flooding, extreme heat, hurricanes, and earthquakes. Enhancing the durability and longevity of concrete structures remains a key challenge, highlighting the need for resilient concrete materials.

Understanding concrete's long-term performance in various environments is critical for sustainable infrastructure. Durability directly affects the lifespan of concrete structures, necessitating materials with enhanced durability performance [3]. Traditional concrete, however, lacks sufficient durability due to its low tensile resistance, high susceptibility to cracking, and thus high vulnerability to water and chloride ingress, leading to steel corrosion and durability issues such as freeze–thaw damage [4]. Conversely, ultra-high-performance concrete (UHPC) offers high compressive strength (>150 MPa), low porosity, low permeability due to enhanced matrix density, and good ductility through the inclusion of fibers [5–14]. Consequently, UHPC effectively addresses durability challenges, mitigates infrastructure aging, and extends life expectancy [3].

While UHPC provides improved infrastructure durability, research on its long-term durability is limited compared to normal concrete due to its slower deterioration rate. Several research institutes have initiated durability research on non-proprietary UHPC in terms of permeability, drying shrinkage, F–T resistance, and absorption.

Permeability in UHPC is typically assessed using rapid chloride permeability tests (RCPT) or electrical resistivity, which are inversely related. Studies have reported electrical surface resistivity values of 6–500 $\text{k}\Omega\cdot\text{cm}$ after 28 days of curing in non-proprietary UHPCs [15–18]. The FHWA found that proprietary UHPCs had resistivity ranges of 40–450 $\text{k}\Omega\cdot\text{cm}$ and 50–900 $\text{k}\Omega\cdot\text{cm}$ after 28 and 56 days, respectively [19], and uniaxial resistivity values of 2.2–8 $\text{k}\Omega\cdot\text{cm}$ and 4–10 $\text{k}\Omega\cdot\text{cm}$ after the same periods [20]. However, some values did not meet the low chloride ion penetration threshold of 21 $\text{k}\Omega\cdot\text{cm}$ after 28 days [21].

Prior studies also showed that drying shrinkage ranged from 0.01–0.12% with fibers [15,16,22–25] and 0.05–0.14% without fibers [18,26] after about 100 days. Likewise, autogenous shrinkage ranged from 0.03–0.062% with fibers [16,23,26,27] and 0.10–0.14% without fibers [18,24] after a month. The FHWA found that proprietary UHPCs with fibers had drying shrinkage of 0.03–0.13% and autogenous shrinkage of 0.02–0.09% after 180 days with different curing conditions. All these results consistently indicate that fibers reduce both types of shrinkage [3,16,19,22,23,25–28] and that curing conditions also significantly affect the shrinkage behavior [27,28].

Previous studies on non-proprietary UHPC show a wide range of relative dynamic modulus of elasticity (RDME) values, ranging from 93–110% after 300 F–T cycles [15,18,22,27] to 90–100% after 350 F–T cycles [29], over 100% after 600 F–T cycles [16], 95–98% after 1098 F–T cycles [30], around 100% after 60 F–T cycles [4], and 97–103% after 96 F–T cycles [31]. Proprietary UHPCs were observed to have RDME values of 96–112% [28], about 100% after 600 F–T cycles [19], and 100–102% after 300 F–T cycles [32]. The variability in RDME indicates that F–T damage is influenced by curing conditions, sample sizes, type, and the proportion of UHPC constituents used.

Although there has been research on UHPC durability, findings are often documented in reports rather than peer-reviewed journal papers. Most studies cover short study periods, typically a few months, with limited data points, which are insufficient to capture the transient behavior of durability properties. UHPC, however, deteriorates slowly, indicating its potential to withstand aggressive environments over long periods. Few studies have determined the service life of UHPC structures and compared them with those made with normal or high-strength concrete. Additionally, no techno-economic analysis (TEA) of UHPC technology has been performed, despite the Department of Energy (DOE) mandates that require TEA for every emerging technology or new product before considering them

in service. TEA considers product longevity, material costs, embodied carbon, and energy. Therefore, to fill this knowledge gap, it is crucial to study UHPC durability over extended periods, determine service life, and perform TEA. These will help to promote UHPC technology in the construction industry.

Non-proprietary UHPCs, developed using locally available materials, save cost and energy and reduce carbon footprint (CFP). Various research institutes and agencies, such as departments of transportation (DOT), have created these cost-effective and design-flexible UHPC mixes [33]. The authors previously developed environmentally friendly non-proprietary UHPC mixtures with local materials, achieving over 150 MPa (22 ksi) compressive strength and 280–330 mm workability without special treatments [33]. This research provides a comprehensive study of critical durability properties, including absorption, electrical surface resistivity, shrinkage, and freeze–thaw, in promising non-proprietary UHPCs. Additionally, this research provides information about (i) longer study periods, (ii) durability properties that are used to determine service life, (iii) the TEA of non-proprietary UHPCs, and (iv) a durability assessment factor to evaluate UHPC durability. To the authors’ best knowledge, these concepts are novel in UHPC research.

2. Material Constituents and Mix Designs

2.1. Properties of Material Constituents

Ordinary Portland cements type II/V (C II/V) were selected for their low C_3A content (4%), which was less than 8%, as per the recommendation from Sakai et al. 2008 [34], satisfying ASTM C150 [35], as well as its low Blaine value of 401 m²/kg. Grey un-densified silica fume (SF) was selected based on median particle size ($D_{50} = 0.5 \mu\text{m}$) and low carbon content (0.3%). Fly ash class C (FA C), containing calcium oxide (CaO) greater than 18%, as per ASTM C618 [36], was selected with median particle size ($D_{50} = 11.3 \mu\text{m}$). Recycled glass powder (RGP) of type GS was used based on its local availability and its favorable particle size ($D_{50} = 9.4 \mu\text{m}$), which fills the particle size gap between SF and cement. Similarly, based on activity test, grade 120 GGBS with median particle size ($D_{50} = 11.6 \mu\text{m}$) was used in this research. Only one type of local aggregate, basalt sand (B), with a maximum particle size of 1.18 mm was included here. It was sieved to follow the modified Andreasen and Andersen (A&A) curve with a q-value of 0.37 for optimum packing density. Regarding fiber (F) reinforcement, striated, round steel fibers of either 13 mm or 19 mm in length and 0.2 mm in diameter were used. Their tensile strength is 2850 MPa and modulus of elasticity is 210 GPa.

The summary of these UHPC constituents is presented in **Table 1**.

Table 1. Summary of UHPC constituents used in this research.

Ingredients	IDs	Properties	Size
OPC type II/V	CII/V	Moderate fineness, low C_3A , high $C_3S + C_2S$	$D_{50} \approx 25 \mu\text{m}$
Silica fume	SF	Un-densified grey SF with low carbon content	$D_{50} = 0.5 \mu\text{m}$
Recycled glass powder	RGP	Type GS, white color	$D_{50} = 9.4 \mu\text{m}$
Fly ash C	FA C	Type C	$D_{50} = 11.3 \mu\text{m}$
Granulated glass blast-furnace slag	GGBS	Grade 120	$D_{50} = 11.6 \mu\text{m}$
Basalt sand	B	High strength and high modulus of elasticity, low water absorption	Less than 1.18 mm (modified A&A, $q = 0.37$)
High range water reducer	HRWR	Solid content = 29%	
Fiber	F	Smooth and straight steel fiber	13 mm and 19 mm long/0.2 mm dia.

Furthermore, detailed chemical compositions of cementitious materials are presented in **Table 2**.

Table 2. Chemical compositions of cementitious materials (based on data from suppliers).

Compositions	C	SF	RGP	FA C	GGBS
SiO ₂ (%)	20.3	96.1	72.8	34.3	
Al ₂ O ₃ (%)	4.00	4.40	1.74	18.1	11.4
CaO (%)	63.1	0.16	14.0	27.3	
MgO (%)	2.70	0.20	0.67	5.88	
K ₂ O (%)		0.37		0.38	
Fe ₂ O ₃ (%)	3.60	0.18		5.70	
Na ₂ O (%)		0.03	10.9	1.81	
SO ₃ (%)	3.00	0.12		2.11	0.10
CO ₂ (%)	1.60				
CaCO ₃ (%)	86.0				
Cl (%)		0.03			0.01
S (%)				2.06	0.80
Equivalent alkalis (%)		0.27			0.50
Specific gravity	3.15	2.22	2.46	2.76	2.95

2.2. UHPC Mix Designs

This research is a continuation of the previous work on designing resource-efficient, non-proprietary UHPCs using local materials [33]. The most promising UHPC matrices from prior research, known for their high resource efficiency factor (*R*), were selected for the durability investigation [33]. A resource efficiency factor (*R*) was calculated for each UHPC mixture based on the relative workability, compressive strength, cost, and carbon footprint (*CFP*), as per [33].

The reference UHPC matrix (**M**) uses ordinary Portland cement type II/V (PC II/V), un-densified gray silica fume (SF), fly ash class C (FA C), basalt (B), and high-range water reducer (HRWR). The weight proportions of ingredients for reference mix were kept as 1:0.25:0.25 (PC: SF: FA C) with an aggregate to cement ratio by weight (*A/C*) of 0.8, an aggregate to powder ratio (*A/P_o*) of 0.53, and a solid content of HRWR to cement ratio of 0.011. The water added to the mix accounts for the superplasticizer's water content and the aggregate's water absorption. The actual mixing water excludes the absorbed water within the aggregate and the water contributed by the superplasticizer.

In addition to reference matrix (**M**), the other three most-promising matrices were investigated:

M-0.2SF: Uses 20% of SF by weight of cement (bwoc) instead of 25%, optimizing compressive strength and workability [33]. The *W/C* was reduced from 0.24 to 0.23 to maintain comparable spread values.

M-RGP: FA C was replaced by RGP in M-0.2SF. The *W/C* increased from 0.23 to 0.25 due to RGP's higher water demand. It uses 20% SF bwoc, to compare directly with M-0.2SF.

M-0.3GGBS: Replaces 30% of cement weight with GGBS in the mix M, maintaining other proportions for optimal performance [33].

Additionally, four fiber-reinforced UHPCs with striated fibers of 19 mm and 13 mm at fiber volume fractions (*V_f*) of 1% and 1.5%, respectively, were selected. The *W/C* ratios were 0.24 for two and 0.29 for the other two fiber-reinforced UHPCs, achieving desired

spread values of 280–340 mm, as recommended by Wille et al. [37], measured according to ASTM C 230/230M [38].

Table 3 provides an overview of the optimized UHPC matrices and fiber-reinforced UHPCs, selected for durability investigation.

Table 3. Overview of optimized UHPC matrices.

IDs	UHPC Matrices				Fiber-Reinforced UHPC			
	M	M-0.2SF	M-RGP	M-0.3GGBS	M-1.0%	M-1.5%	M-1.0%	M-1.5%
C II/V (kg/m ³)	924	965	925	641	717	714	751	747
GGBS (kg/m ³)				275				
SF (kg/m ³)	231	193	193	229	179	179	188	187
SCMs (FA C)	231	241	241 (RGP)	229	179	179	188	187
C:SF:SCM *	1:0.25:0.25	1:0.20:0.25	1:0.20:0.25	(0.7 + 0.3 **):0.25:0.25		1:0.25:0.25		
Total binder (kg/m ³)	1386	1399	1387	1375	1076	1071	1126	1120
Water (kg/m ³)	207	206	207	205	203	202	168	167
HRWR (kg/m ³)	34	36	34	34	27	27	28	28
B (kg/m ³)	747	747	747	747	1076	1070	1126	1120
F (kg/m ³)	-	-	-	-	79	118	79	118
W/C	0.24	0.23	0.25	0.24	0.29	0.29	0.24	0.24
A/C	0.81	0.77	0.81	0.81	1.5	1.5	1.5	1.5
Spread (mm)	282	305	268	301	278	265	253	245
R-factor	1.36	1.44	1.26	1.41	1.32+	1.25+	1.67+	1.20+

* Supplementary cementitious material, ** GGBS, +Based on cost boundary value: cost_b = USD 1000/m³.

3. Methodology

3.1. UHPC Mixing

A 20-quart Hobart mixer with bi-rotary motion was used to prepare all UHPCs, following the established UHPC mixing protocol [39]. Mixing occurred at three speeds to ensure the proper dispersion and breakdown of agglomerations.

Initially, silica fume (SF) and aggregates were dry mixed for five minutes at speed one (107 rpm) to break down agglomerations and minimize airborne particles. Portland cement (PC) and supplementary cementitious materials (SCMs) were then incorporated and mixed for an additional five minutes at the same speed to ensure even distribution.

Water and one-third of the high-range water reducer (HRWR) were gradually introduced over one minute, followed by the remainder of the HRWR, all at speed one. The mixer speed was then increased to speed two (198 rpm) to enhance particle dispersion by applying higher shear forces, effectively breaking any remaining agglomerations. After turnover, mixing continued for 5 min at speed two, followed by a final 30-second mix at speed three. When fibers were added, the speed was reduced to one, and mixing proceeded for two additional minutes to ensure uniform fiber dispersion.

3.2. Sample Preparation

For each mix, two large beams ($76 \times 10 \times 406$ mm), two small prisms ($25 \times 25 \times 305$ mm), and two cylinders (76×152 mm) were cast for F–T, shrinkage, and electrical surface resistivity testing, respectively. Samples were consolidated by vibrating for 2.5 min at a frequency of 3.5 Hz. About 30 min after casting, the molds were covered with plastic to avoid water evaporation. F–T beams and cylinders were cured in water at room temperature and RH greater than 95%, while small prisms were air-dried at room temperature with RH below 50% after demolding. After 14 days of curing, F–T beams were transferred to the F–T chamber.

3.3. Absorption

The water absorption test was based on ASTM C642 [40]. Samples were cut into 2-inch slices and submerged in water for 48 h. They were then prepared to achieve a saturated surface-dry (SSD) condition, weighed, and oven-dried at 100 °C for 48 h. After cooling for one hour, the samples were reweighed and oven-dried for an additional day. The following day, the samples were weighed again. If the weight change was less than 10% compared to the previous day, the drying process was concluded.

3.4. Electrical Surface Resistivity

Concrete surface resistivity tests were conducted to determine concrete permeability according to AASHTO TP95 [41]. The conceptual drawing and the test setup are shown in **Figure 1**. This non-destructive method monitors changes in resistivity over time, indicating the development of the concrete pore structure [42]. The cylinders were demolded one day after casting and cured at room temperature and greater than 95% RH. The test setup was modified to adjust $3'' \times 6''$ cylinders since it was originally designed for $4'' \times 8''$ cylinders. Two cylinders from each UHPC series were used to measure electrical surface resistivity. Measurements were taken at 0°, 90°, 180°, and 270° on each cylinder. The average of these four measurements per cylinder was calculated to obtain a representative value for each UHPC series. Resistivity was measured daily for the first months, every other day thereafter, and then weekly until the equipment's detection limit was reached.



Figure 1. Electrical surface resistivity testing on $3'' \times 6''$ cylinders.

3.5. Shrinkage

Shrinkage is a major cause of cracking in hot and humid weather. To measure shrinkage, two small prisms ($25 \times 25 \times 305$ mm) were prepared for each series, as shown in **Figure 2**. A length comparator was used to measure long-term shrinkage according to ASTM C157/C157M-17 [43]. Each beam had shrinkage studs on both sides and was demolded after one day, then kept at room temperature with relative humidity (RH) below 50%. Shrinkage measurements were taken daily for the first month, every other day subsequently, and then weekly to observe both transient and steady-state behavior over the years. After one year, the samples were cured normally to observe any changes in length.

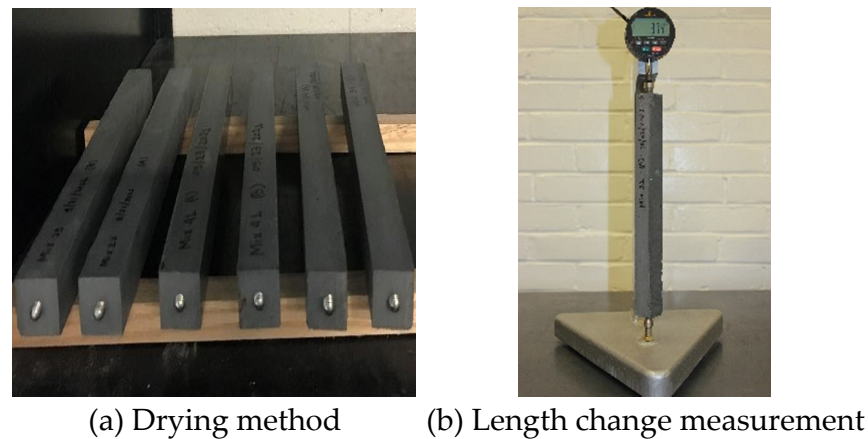
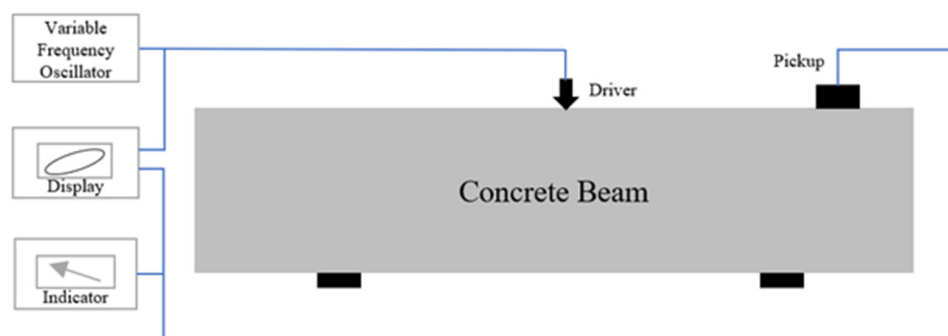


Figure 2. Drying shrinkage.

3.6. Freeze–Thaw Durability

Freeze–thaw (F–T) testing is a widely recognized and mandated practice in the construction industry to evaluate concrete’s resistance to F–T degradation, serving as an indirect assessment of material durability.

Two beams from each mix series were used for F–T testing, as per ASTM C666 [44]. These beams were cured normally for 14 days and subjected to 600 F–T cycles, exceeding the ASTM requirement of 300 cycles. The samples were fully submerged in water-filled containers slightly larger than sample size to reduce water volume as less water requires less time and energy to complete a cycle. The F–T chamber, as shown in **Figure 3**, facilitated thermal expansion and contraction during the cycles. The chamber was programmed to execute freezing and thawing cycles every 3 hours, with temperature limits of $-23\text{ }^{\circ}\text{C}$ and $2\text{ }^{\circ}\text{C}$ ($-9\text{ }^{\circ}\text{F}$ and $36\text{ }^{\circ}\text{F}$), respectively. The resonance frequency and weight of beams were measured after every 30 cycles, up to 600 F–T cycles.



(a) Conceptual drawing of resonance frequency measurement



(b) Test setup for frequency measurement



(c) F–T chamber

Figure 3. Freeze–thaw chamber and resonance frequency measurement setup.

The forced resonance frequency method was used to measure the resonance frequency of UHPC beams, as per ASTM C215 [45]. An electro-mechanical driving unit forced the beam to vibrate, and a lightweight pickup unit monitored the response. The driving unit, activated by a variable frequency oscillator, generated vibrations that varied in resonance, depending on the concrete quality. The driving frequency was adjusted until the measured response reached maximum amplitude. The frequency at which the maximum response occurred was recorded as the resonance frequency of the specimen.

$$\text{Dynamic modulus of elasticity, } E_{dyn} = CMn^2 \quad (1)$$

where M = mass of specimen (kg), n = fundamental transverse frequency (Hz), and

$$C = 0.9464 \times \frac{L^3 T}{bt^3} \quad \text{m}^{-1} \text{ for a prism} \quad (2)$$

where L , t , and b are the length, width, and height, respectively, of the prism in meters.

The relative dynamic modulus of elasticity (RDME) was calculated as the ratio of the dynamic modulus at the n th F–T cycle to its initial value at zero F–T cycles:

$$RDME = \frac{n_1^2}{n^2} \times 100\% \quad (3)$$

where n and n_1 are the fundamental transverse frequency after zero and n F–T cycles, respectively.

The summary of the test details used in this research is presented in **Table 4** below.

Table 4. Summary of tests and their respective standards.

Tests	Specimen Dimension	Standards
Water absorption test	50 mm thick disk of 76 mm diameter	ASTM C642 [40]
Electrical surface resistivity	Cylinder of 76 mm diameter and 152 mm height	AASHTO TP95 [41]
Total shrinkage	Beam of 25 mm × 25 mm × 305 mm	ASTM C157 [43]
Freezing and thawing durability	Beam of 76 mm × 102 mm × 406 mm	ASTM C666 [44]

4. Test Results and Discussion

4.1. Absorption

The water absorption of 28-day UHPCs is presented in **Figure 4**. All four UHPCs exhibited absorption rates of less than 1.4%, which is remarkably low. High-strength concrete usually has water absorption of 2–5% in 28 days [46,47], and concrete is considered durable if it is less than 5% [47]. Two commercial UHPCs (denoted as Comm. 1 and Comm. 2 in **Figure 4**) were tested for comparison, and the results indicate that their absorptivity is comparable to the non-proprietary UHPCs. This very low water absorption is attributed to their dense microstructure and disconnected capillary pores [17,48,49].

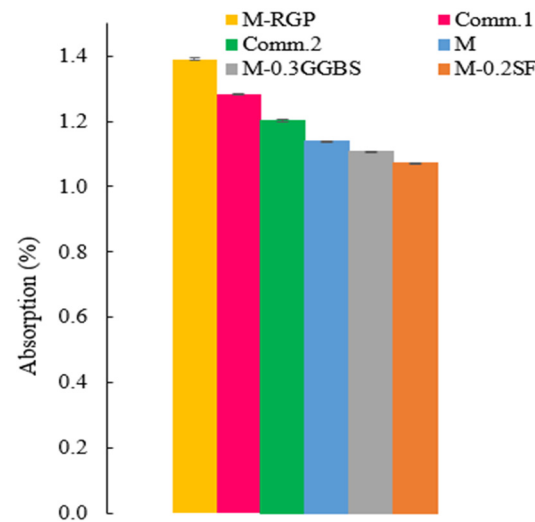


Figure 4. Absorption and its correlation with air content in UHPC.

4.2. Electrical Surface Resistivity

The electrical surface resistivity test results of UHPCs are shown in **Figures 5 and 6**. The purple dashed lines in the figures represent the threshold for low ion penetration of 21 kΩ·cm and negligible ion penetration of 254 kΩ·cm [41], while the purple solid line indicates the instrument's detection limit. The experimental data suggests that a second-degree polynomial equation fits the resistivity data well:

$$\text{Electrical surface resistivity} = at^2 + bt + c \quad \text{in k}\Omega\cdot\text{cm} \quad (4)$$

where t is time (days for normal curing or hours for steam curing) and a, b, c are fitting parameters (values provided in each graph). The fitted equations show a strong correlation between resistivity values and curing times, with R-squared values exceeding 0.98 for all graphs.

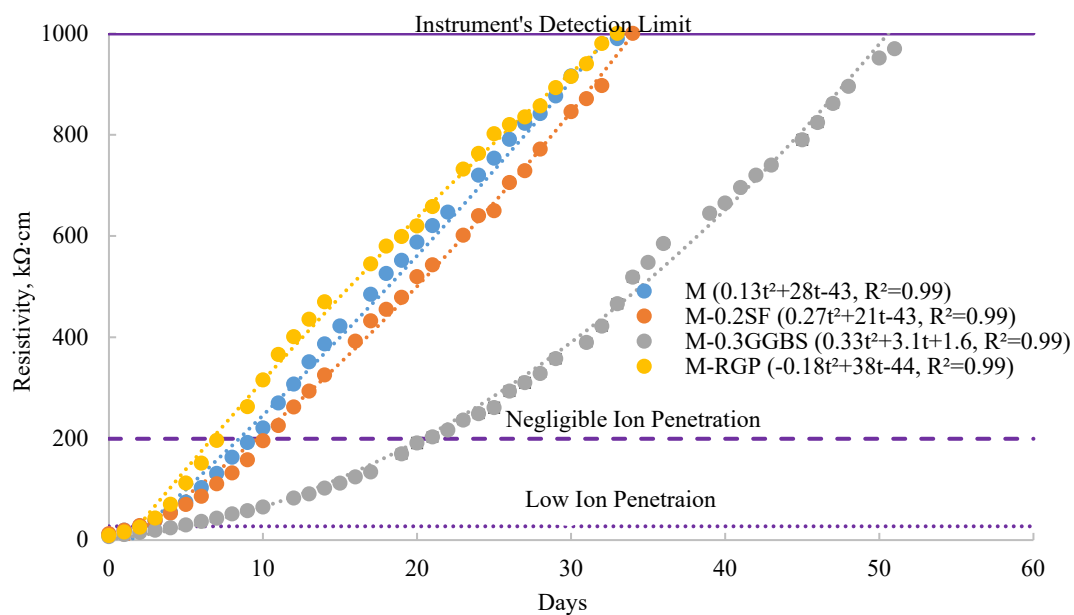


Figure 5. Electrical surface resistivity of newly developed non-proprietary UHPC paste.

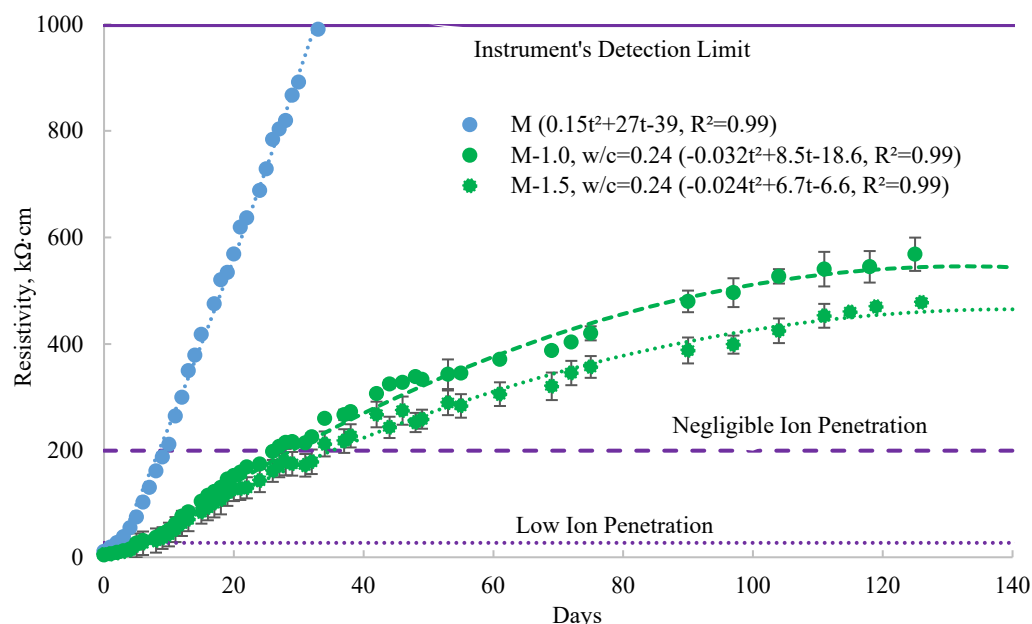


Figure 6. Electrical surface resistivity of newly developed non-proprietary UHPC paste.

4.2.1. Resistivity of UHPC Matrices Without Fibers

Figure 5 presents the electrical surface resistivity values of all four UHPC matrices without fibers, along with fitting parameters for **Equation (4)**. M-RGP, M-0.2SF, and M exceeded the low ion penetration threshold after 2 days and negligible ion penetration threshold in fewer than 10 days. In contrast, M-0.3GGBS required 4 days and 21 days, respectively, to reach these thresholds. All UHPC matrices displayed similar electrical surface resistivity to M-0.3GGBS, as GGBS takes time to develop a denser microstructure. At any given time, M-0.3GGBS showed up to 75% of the electrical surface resistivity of reference mix M. GGBS delays hydration reaction in early age stages by reducing the maximum temperature rise, which suppresses the early formation of CSH, but increases its formation in the later stages [50,51]. The delay is due to the slow dilution of clinker content in the paste because of the limited pozzolanic activity of the metals [51]. Despite these differences, all UHPC matrices eventually reached the equipment's detection limit of 1000 kΩ·cm at their own time.

Compared to M, M-0.2SF achieved electrical surface resistivity relatively slowly, generally showing 14% less resistivity on any given measurement day.

Similarly, M-RGP exhibited a rapid increase in electrical surface resistivity compared to M and M-0.2SF, due to the presence of RGP instead of FA. This trend is consistent with steam curing as well [52]. Overall, M-RGP achieved up to 30% and 60% greater electrical resistivity compared to M and M-0.2SF, respectively, on any measurement day, although this increase slowed after a month.

4.2.2. Effect of Fiber Volume Fraction on Resistivity

Figure 6 presents the electrical surface resistivity of UHPCs with V_f of 1% and 1.5% at W/C of 0.24, the same as that of UHPC matrices without fibers. The UHPCs with fibers are denoted as M-1.0-0.24 and M-1.5-0.24 for V_f values of 1% and 1.5%, respectively. The W/C and A/C ratios were kept constant to enable a direct comparison of UHPC with and without fibers, highlighting the effect of steel fiber content on resistivity measurements. The measured resistivities of M-1.0-0.24 and M-1.5-0.24 are about 70–90% and 75–95% lower than those of M at comparable concrete ages. Previous research also found an up to 70% reduction in electrical resistivity in fiber-reinforced conventional concrete compared to plain conventional concrete [53]. This significant reduction in resistivity, despite having

the same matrix with the same W/C and A/C ratios, demonstrates the influence of steel fibers on resistivity measurements. Steel fibers being conductive bridges the cementitious matrix and create pathways for electrical charges, resulting in an artificial decrease in electrical resistivity [54]. The increase in V_f indicates a higher number of steel fibers, which contributes to the overall increased conductivity of the specimen, thereby decreasing the electrical surface resistivity [55].

4.2.3. Effect of W/C on Resistivity

Figure 7 shows the electrical surface resistivity of UHPCs with W/C ratios of 0.24 and 0.29, represented by green and blue lines, respectively. Larger and smaller dashed lines denote UHPCs with V_f of 1.0% and 1.5%, respectively. An increase in W/C from 0.24 to 0.29 resulted in a resistivity decrease of 20–65% for M-1.0 and 25–80% for M-1.5. This indicates that a higher W/C ratio produces a more porous and permeable concrete matrix [56]. The increased porosity facilitates ion conduction within the concrete matrix, thereby increasing overall conductivity and reducing electrical resistivity.

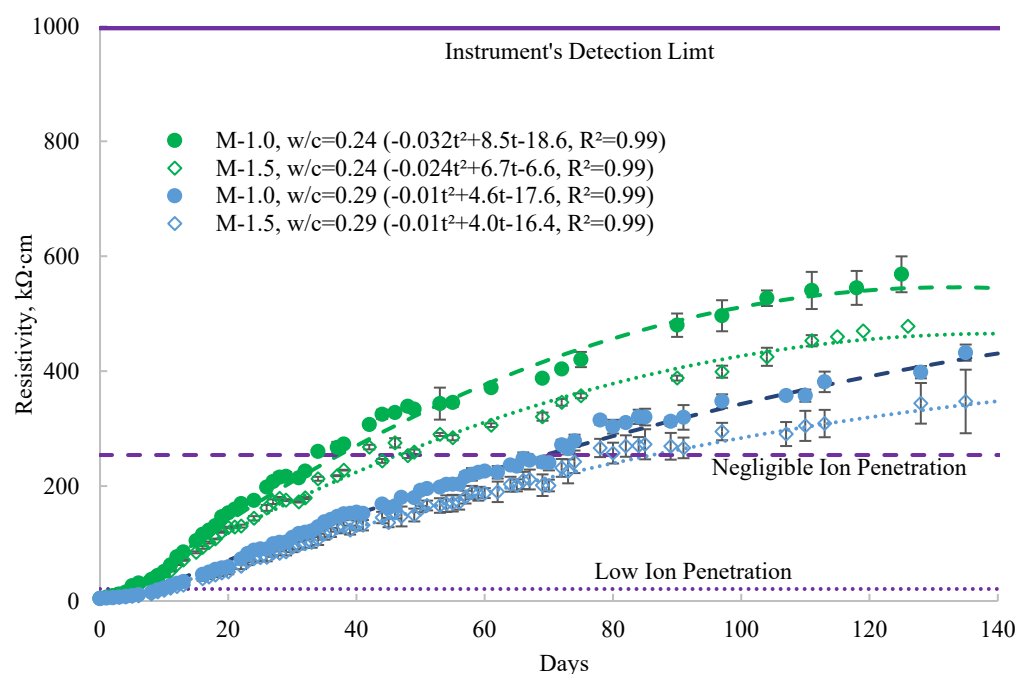


Figure 7. Electrical surface resistivity of newly developed non-proprietary UHPC paste.

4.2.4. Relationship Between Compressive Strength and Resistivity

Figure 8 illustrates the relationship between the electrical surface resistivity and compressive strength of non-reinforced, non-proprietary UHPCs at various curing ages. Measurements for both compressive strength and electrical surface resistivity were taken on the same day and under identical curing conditions. The empirical relationship between compressive strength and electrical surface resistivity, derived from the collected data, is expressed by the following formula:

$$\rho = 3.26 \times e^{0.03 \times f_c'} \quad (5)$$

where ρ is the electrical surface resistivity in $\text{k}\Omega \cdot \text{cm}$ and f_c' is the compressive strength in MPa. The strong correlation between these parameters is evidenced by an R-squared value of 0.88. Both electrical resistivity and compressive strength are linked to the density of the concrete matrix. This correlation suggests that improvements in matrix density positively influence both parameters.

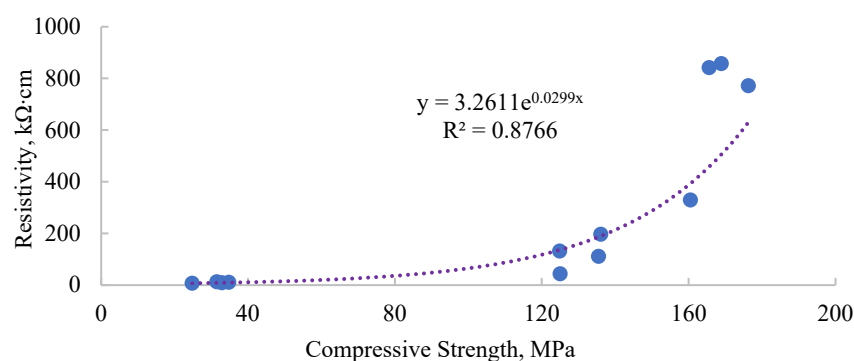


Figure 8. The relationship between electrical surface resistivity and compressive strength.

4.2.5. Comparison of Electrical Resistivity with Other Researchers

The highest 28-day electrical surface resistivity values from this research are compared to the highest values reported in previous studies, as discussed in the Introduction. **Figure 9** presents a summary of electrical surface resistivity results from previous researchers [15–19]. Green dots represent the test results from non-proprietary UHPCs, while orange and yellow dots represent proprietary UHPCs. FHWA-A to FHWA-E refers to electrical surface resistivity values and FHWA-a to FHWA-e refers to uniaxial resistivity values. This figure clearly shows that almost all UHPCs achieved the low ion penetration threshold. However, only three studies, including the present one, achieved negligible penetration threshold within 28 days. Most previous studies did not measure electrical resistivity continuously, as was measured in this current research.

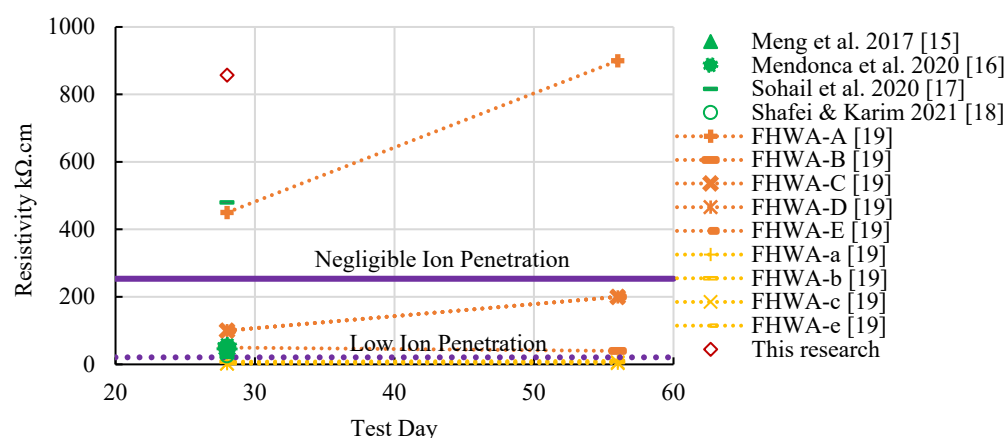


Figure 9. Summary of electrical resistivity results.

4.3. Drying Shrinkage

In this study, drying shrinkage is investigated in UHPCs with and without fibers. Based on the experimental data, shrinkage can be predicted using **Equation (6)**, which is best suited with the appropriate fitting parameters:

$$\text{Shrinkage, } \epsilon(t)\% = \frac{t}{100 \times (\alpha + \beta t)} \quad (6)$$

where α, β are fitting parameters provided in the graphs and t refers to the age of the concrete sample. The value of α determines the initial steepness of the shrinkage curve, while β determines the curvature in the subsequent stages.

4.3.1. Drying Shrinkage in UHPC Matrices

Figure 10 shows the drying shrinkage behavior and mass loss over time for all promising UHPCs dried at room temperature and below 50% RH. Shrinkage and mass change, both associated with moisture loss, are presented in **Figure 10a,b**, highlighting their interrelation. These figures also show the prediction model with suitable fitting parameters alongside the experimental data. **Figure 10a** shows a steep drying shrinkage curve for up to one and half months, transitioning to an almost horizontal curve after reaching a steady state by two months. All UHPC matrices achieved a drying shrinkage of less than 0.13% within two months, with no significant change observed even after a year.

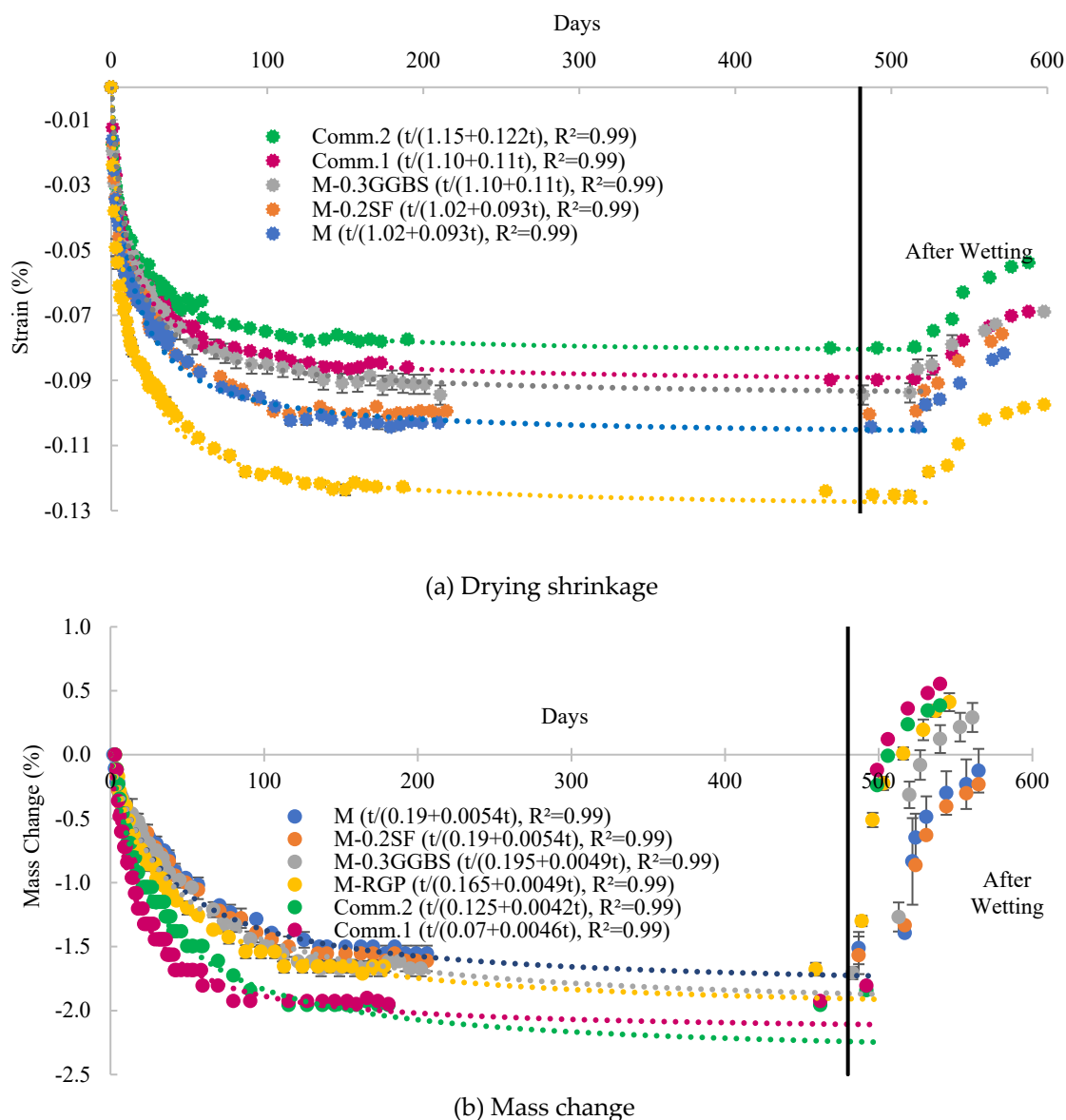


Figure 10. Drying shrinkage and mass change of UHPC matrices.

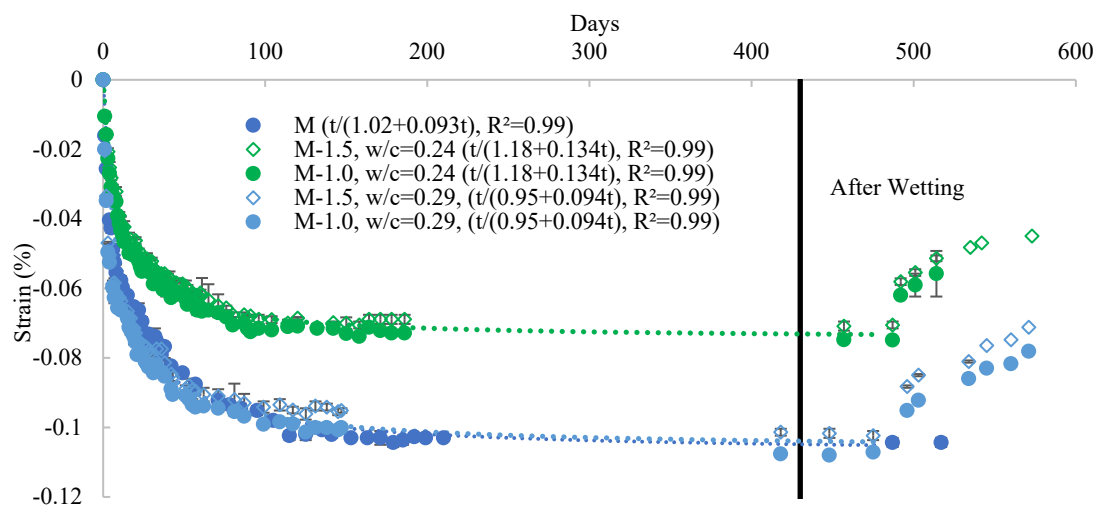
Compared to the reference mix M, M-0.2SF exhibited up to 4% less strain and M-0.3GGBS had up to 16% less strain, while M-RGP showed up to 30% more strain. Research suggests that increasing GGBS replacement reduces both autogenous [57–59] and drying shrinkage slightly [57,60], as GGBS tends to maintain water content due to its hygroscopicity properties [60]. M-RGP showed a relatively higher shrinkage compared to other UHPC matrices due to its higher W/C ratio (0.25 vs. 0.24). Drying shrinkage is sensitive to W/C ratio, with higher ratios resulting in greater shrinkage [49]. Both commercial UHPCs

showed less strain: Comm.1 exhibited up to 12% less strain and Comm.2 up to 23% less strain compared to M.

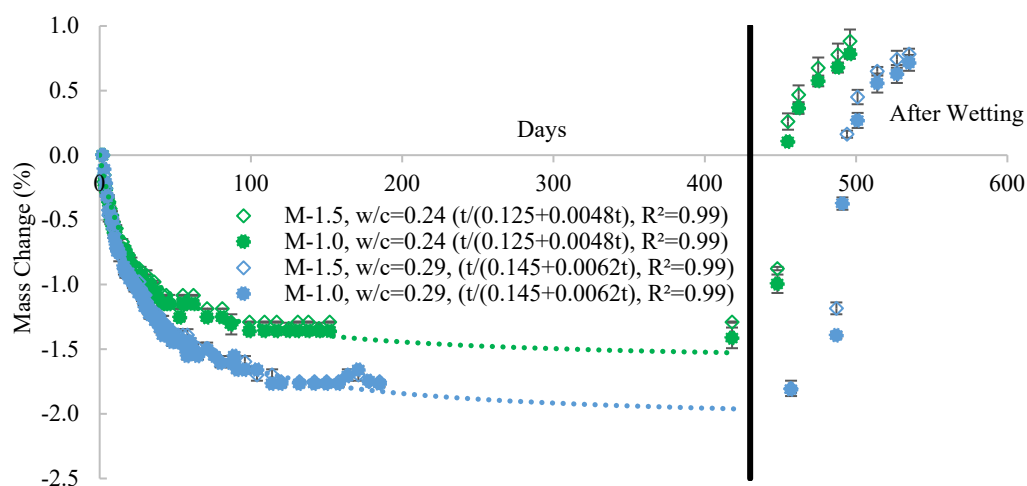
Unlike shrinkage, the mass loss graph remained steeper even after two months, becoming negligible after three months. This extended period to reach a steady state is due to the ongoing hydration reaction utilizing available moisture and loss due to evaporation. Continuous hydration reduces pore size, increasing the surface tension among water molecules in finer voids, making moisture loss more difficult. Thus, the mass loss curve required more time to stabilize into a horizontal trajectory.

4.3.2. Drying Shrinkage in Fiber-Reinforced UHPCs

Figure 11a compares the drying shrinkage of fiber-reinforced UHPCs at W/C ratios of 0.24 and 0.29 with the UHPC matrix (M) at W/C = 0.24. There is a significant increase in shrinkage, up to 42%, when the W/C ratio increases from 0.24 to 0.29. A higher W/C ratio results in a less dense microstructure and increased water evaporation from the concrete, leading to greater shrinkage.



(a) Drying shrinkage



(b) Mass change

Figure 11. Drying shrinkage and mass change of fiber-reinforced UHPCs.

Figure 11a also shows that the use of fibers in UHPC at the same W/C ratio can reduce shrinkage. Both fiber-reinforced UHPCs showed less drying shrinkage ($\epsilon = -0.0006$ after 150 days) than M ($\epsilon = -0.0009$ after 150 days). The inclusion of fibers decreased shrinkage up to 45%. Increasing V_f from 1.0% to 1.5% further decreased the shrinkage by up to 7% for both the W/C ratios (0.24 and 0.29). Interestingly, the shrinkage of the reference matrix M is similar to that of fiber-reinforced UHPCs, even when the W/C ratio changed from 0.24 (for M) to 0.29 (for M-1.0% and M-1.5%).

Figure 11b shows the mass change in fiber-reinforced UHPC under drying shrinkage. It was observed that the mass change stabilizes after 2.5 months of drying. The mass loss in fiber-reinforced UHPC with a W/C of 0.29 is approximately 30% greater than that with a W/C of 0.24. This indicates that an increase in W/C contributes to greater mass loss in UHPC during drying shrinkage, which further correlates with the drying shrinkage observed in **Figure 11a**.

4.3.3. Shrinkage Behavior of Steam-Cured UHPCs

Figure 12 shows the drying shrinkage behavior and mass loss of all steam-cured UHPCs, which were dried after 24 h of curing. Even during steam curing, all UHPC matrices exhibited shrinkage, as shown in **Figure 12a,b**, but gained mass due to extensive hydration by steam, as seen in **Figure 12c,d**. Compared to **Figures 10 and 11**, steam curing reduced drying shrinkage: M showed up to 26% lower, M-0.2SF up to 22% lower, M-0.3GGBS up to 15% lower, M-RGP up to 40% lower, M-1.0% up to 20% lower, and M-1.5% up to 50% lower strain values. This reduction is likely due to accelerated hydration reaction, which provides a very dense microstructure, leading to mass gain. At the same time, UHPC matrices achieved sufficient strength and stiffness, which restrained the shrinkage. The drying shrinkage caused by moisture loss was hindered due to finer capillary pores with higher surface tension among water molecules, preventing water evaporation from the pores. This is further supported by observations of water-cured UHPCs after a year of drying.

The authors have previously conducted similar studies, concluding that the inclusion of fibers significantly improves shrinkage resistance in steam-cured UHPCs [52].

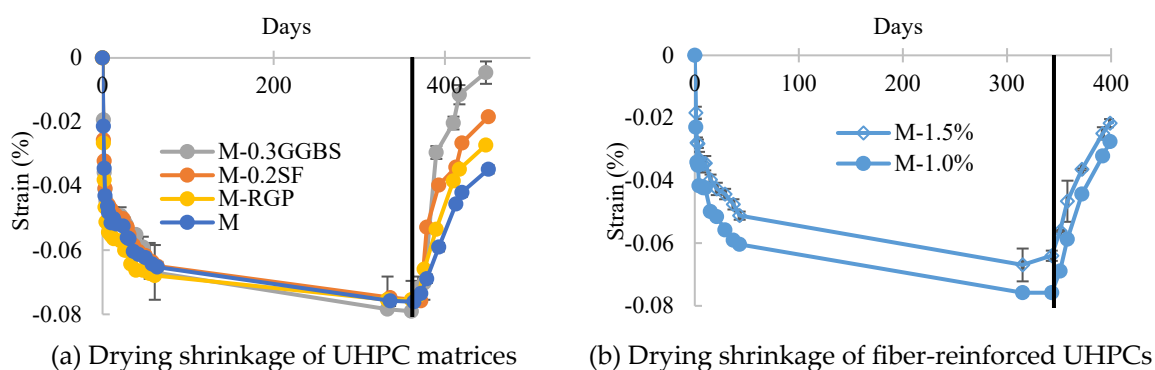


Figure 12. Cont.

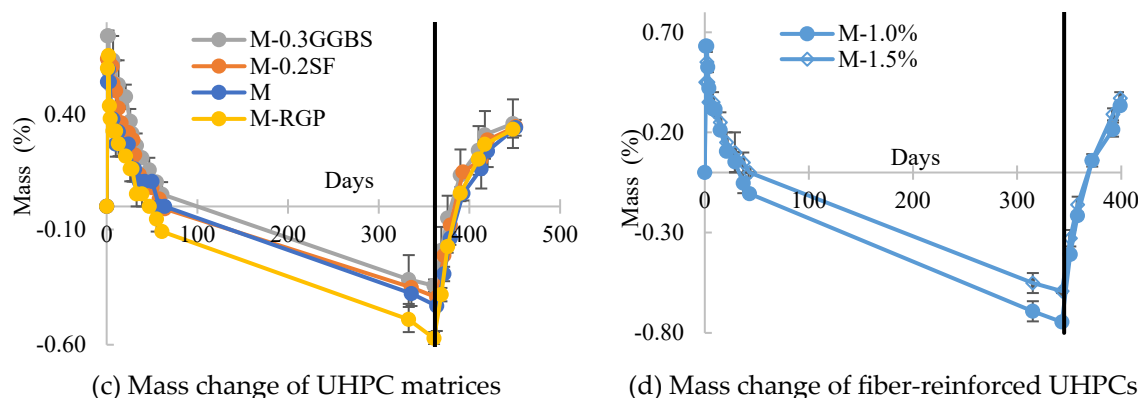


Figure 12. Drying shrinkage and mass change of steam-cured UHPCs.

4.3.4. Comparison of Drying Shrinkage Results with Other Researchers

The lowest drying shrinkage value from this research is compared to those from previous studies, as discussed in the Introduction. **Figure 13** presents a comparative summary of all drying shrinkage results with green dots representing non-proprietary UHPCs and yellow dots representing proprietary UHPCs. The drying shrinkage values from this study are comparable to those in other studies. However, it is important to note that the shrinkage values in this study were measured after almost a year, whereas other studies measured them within a six-months period.

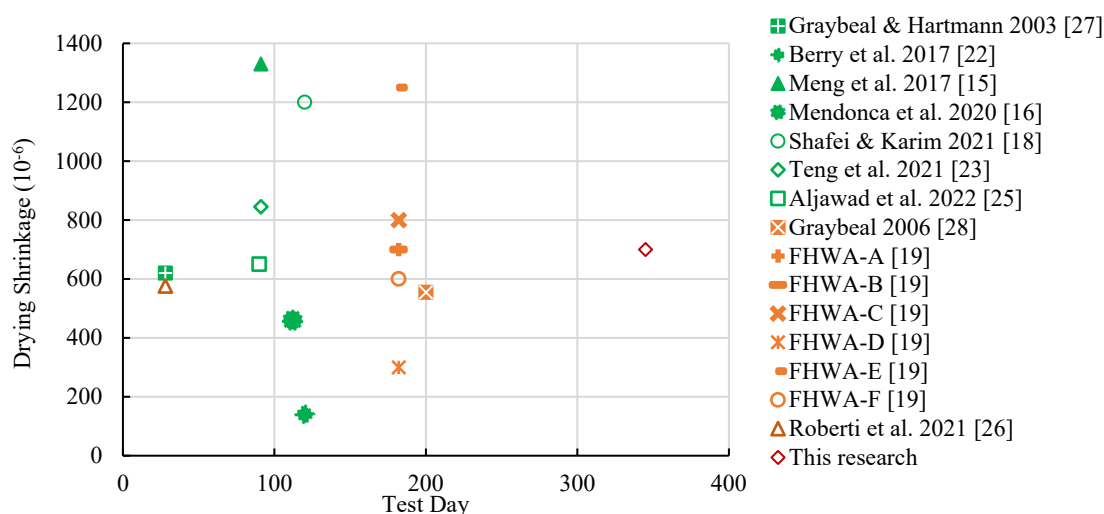


Figure 13. Summary of drying shrinkage of UHPCs.

4.4. Freeze–Thaw Resistance

4.4.1. UHPC Matrices

Figure 14 shows the freeze–thaw (F–T) behavior of newly developed UHPC matrices. Interestingly, there is an increase in the dynamic modulus of elasticity (MOE) and mass, even under extreme F–T cycles. None of the UHPC matrices showed any signs of deterioration after 600 F–T cycles. This resistance to accelerated exposure is attributed to either a very dense microstructure or an efficient internal air void system. The relative dynamic modulus values are 102.3 for M-0.2SF, 103.2 for M-0.3GGBS, 105.1 for M, and 103.5 for M-RGP after 600 F–T cycles. The dynamic MOE is highest for M-0.2SF, followed by M-0.3GGBS, M-RGP, and M. M-0.2SF, which exceeded M by up to 5.9% and M-0.3GGBS by up to 3.3%, while M-RGP acted almost similar to M. Increase in dynamic MOE is supported by the mass gain, which is about 0.1% after 600 F–T cycles.

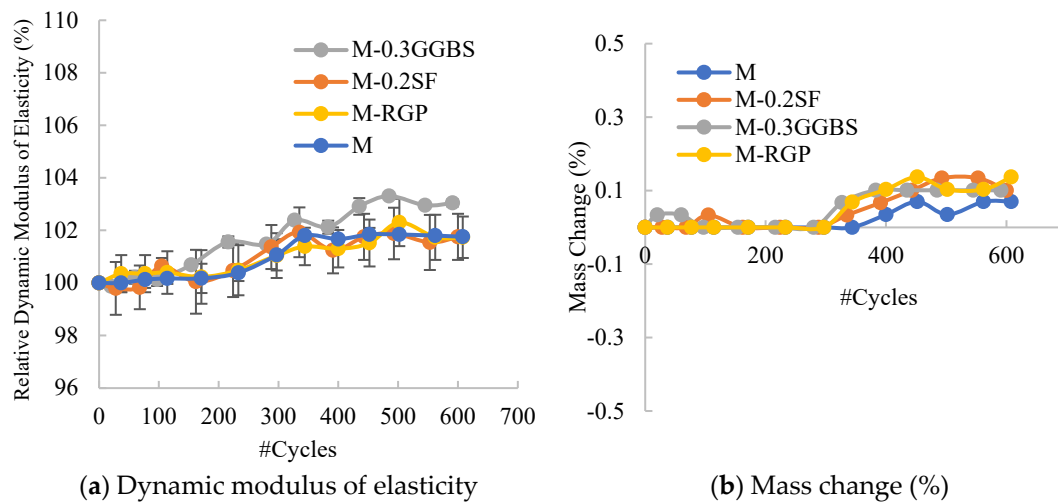


Figure 14. Freeze-thaw resistance of UHPC matrices.

These findings are corroborated by other studies [16]. The mass increase is due to the hydration of unreacted cementitious material during thawing cycles [3,61], where temperatures rise from 0 to 40 °F [44]. Given the very low W/C ratio, not all cementitious material could hydrate during the initial 14-day curing period, continuing to hydrate during the thawing cycles. The formation of a denser microstructure through further hydration leads to an increase in the relative dynamic modulus over time. Such characteristics are typical of self-healing concrete, which can autonomously repair cracks and improve the overall resilience of structures.

4.4.2. Influence of W/C and Fiber Volume Fraction

Figure 15 shows the F–T behavior of fiber-reinforced UHPCs with $V_f = 1.0\%$ and 1.5% , and W/C ratios of 0.24 and 0.29. Similar to UHPC matrices, there is no deterioration even after 600 F–T cycles. All fiber-reinforced UHPCs achieved a RDME greater than 100%: 101.8 for M-1.5-0.24, 102.2 for M-1.0-0.24, 101.4 for M-1.5-0.29, and 101.2 for M-1.0-0.29. The increase in dynamic MOE is supported by the corresponding increase in mass.

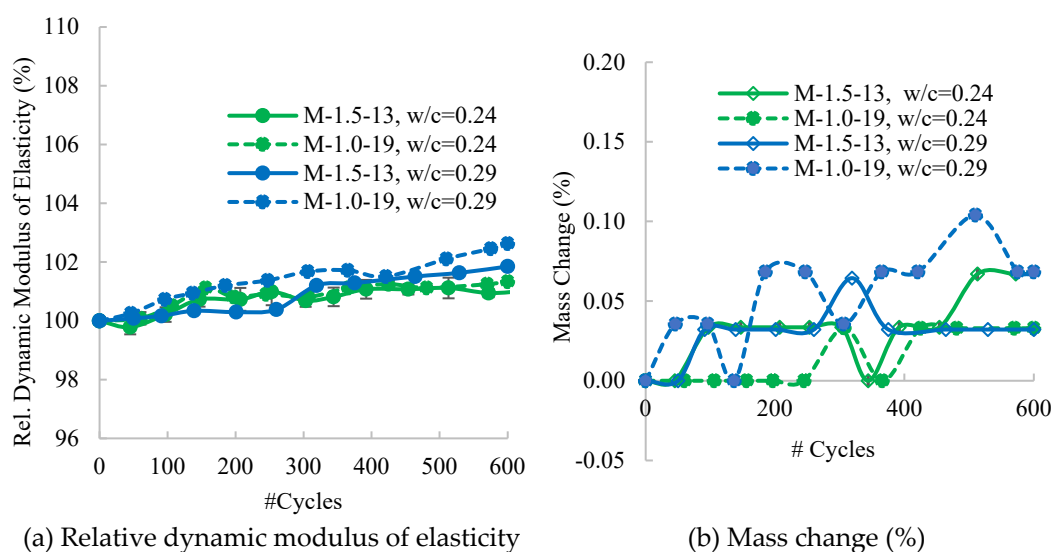


Figure 15. Freeze-thaw resistance of fiber-reinforced UHPCs.

Although M, M-1.5-0.24 and M-1.0-0.24 have the same W/C ratio of 0.24, M-1.5-0.24 and M-1.0-0.24 show higher dynamic MOE, by up to 9.1% and 7.1%, respectively. This

suggests that the addition of fibers might have increased the dynamic MOE. Previous research also demonstrated a 4.5% increase in dynamic MOE with the addition of fibers in UHPC [30].

Furthermore, M-1.0% and M-1.5% showed up to 2.3% and 3.1% increases in dynamic MOE at W/C = 0.24, respectively, compared to W/C = 0.29. This increase is attributed to a denser microstructure of the UHPC matrix influenced by the W/C ratio. However, there is no distinct difference in mass gain among these UHPCs.

4.4.3. Comparison of Freeze–Thaw Results with Other Researchers

The comparative summary of all the freeze–thaw resistance results is presented in **Figure 16**. The green dots represent non-proprietary UHPCs, while yellow dots represent proprietary UHPCs. The RDME is plotted against different F–T cycles. The current study measured RDME after 600 F–T cycles, which is observed to be comparatively higher than those from most previous studies, except one dataset. There is no significant difference between RDMEs from proprietary and non-proprietary UHPCs. The absence of deterioration signs in UHPC specimens, even after 600 freeze–thaw cycles, indicates low water absorption into the sample due to the presence of smaller gel pores, signifying a dense matrix [62].

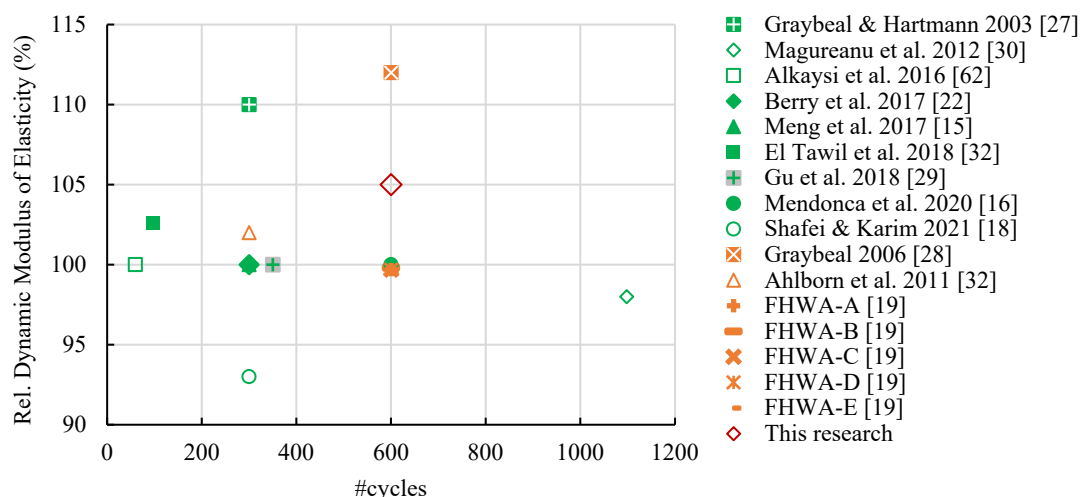


Figure 16. Summary of freeze–thaw resistance of UHPCs from other researchers.

5. Further Implications of Research Findings

5.1. Service Life Modeling from Electrical Surface Resistivity

Several software programs can model the service life of concrete structures, including Life 365, ConcreteWorks (version 2.1.3), and STADIUM (version 2.99). Among these, Life 365 is quite popular. It considers factors such as the percentage of SF and GGBS, as well as the W/C ratio. Other exposure conditions are based on the geographical location of the structures. Life 365 promotes the use of SF and GGBS, as increasing their percentage in the mixture design is said to lengthen the service life of concrete structures. However, this may lead to an overestimation of the service life for concrete mixtures containing SF and GGBS. Additionally, Life 365 was designed to model the service life of structures built with normal concrete and does not account for the specific characteristics of UHPC, such as low W/C ratios (less than 0.25), particle packing principle, fine aggregates, or the use of glass powder or quartz powder. Furthermore, it only considers class FA F. Therefore, this software is insufficient for accurately modeling the service life of a UHPC structure.

Another promising way to determine the service life of concrete is using the error function solution to Fick's second law. According to Fick's second law of diffusion, the

rate at which ions move through a material is proportional to the concentration gradient between the two ends of the material.

Change in concentration per unit time = change in flux per unit distance

$$\frac{\partial C}{\partial t} = -\frac{\partial J}{\partial x} \quad (7)$$

By replacing flux $J = -D \frac{\partial C}{\partial x}$, $\frac{\partial C}{\partial t} = -\frac{\partial}{\partial x} \left(-D \frac{\partial C}{\partial x} \right)$

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2}$$

Using the Gaussian error function in the above equation, the solution becomes

$$C(x, t) = A + B \times \operatorname{erf} \left(\frac{x}{2\sqrt{Dt}} \right) \quad (8)$$

where A and B are constant based on the boundary conditions. The formation factor and self-diffusion coefficient can replace the original diffusion coefficient in the error function solution using the Nernst–Einstein relationship, formulated as follows [63]:

$$\frac{C_{x,t} - C_o}{C_s - C_o} = 1 - \operatorname{erf} \left(\frac{x}{2\sqrt{\frac{t \times D_o}{F}}} \right) \quad (9)$$

where

$C_{x,t}$ = Chloride concentration at depth x (%) = 0.05%

C_o = Initial chloride concentration (%) = 0.02%

C_s = Surface chloride concentration (%) = 0.68%

x = Depth to reinforcement (in) = 2 in cover

D_o = Chloride ion self-diffusion coefficient (in/s²) = 19×10^{-10} m²/s

F = Formation factor

t = Time for exposure limit to be reached (s).

All constants are based on the exposure region. For example, for an urban highway bridge in Bridgeport, Connecticut that carries interstate highways and is exposed to marine conditions, specific values are available.

The formation factor can be determined as per AASHTO PP84 [64]:

$$\text{Formation factor } F = \frac{BR}{PSR} \quad (10)$$

where bulk resistivity (BR) can be calculated from surface resistivity (SR) using an equation provided by Spragg et al. 2013 [65], and pore solution resistivity (PSR) can be obtained from software developed by NIST based on Snyder et al. 2003 [66] with 40% hydration after 28 days or considered as 0.05 $\Omega \cdot m$ for UHPC [63].

$$\frac{SR}{BR} = 1.10 - \frac{0.730}{\frac{d}{a}} + \frac{7.34}{\left(\frac{d}{a}\right)^2} \quad (11)$$

where d and a refer to diameter and probe spacing, respectively. In this study, $d = 76$ mm and $a = 38$ mm.

Thus, this equation becomes

$$BR = \frac{SR}{2.57} \quad (12)$$

Additionally, corrosion rate can be predicted from an empirical formula [67]:

$$V_{corr.} = \frac{301.6}{BR} \text{ mm/year} \quad (13)$$

where BR is in $\Omega \cdot \text{m}$.

After substituting all values into **Equation (9)**, time is calculated in seconds and then converted to years.

Table 5 shows the calculated service life based on **Equations (9) to (12)** and the assumptions described above. The results show that M can last for 376 years, M-0.2SF for 345 years, and M-RGP for 383 years, while M-0.3GGBS only lasts for 147 years. Since GGBS delays the hydration reaction, it takes longer to develop a denser microstructure, resulting in lower electrical surface resistivity at 28 days, which ultimately impacts the service life prediction. This service life can be expected to be even higher, around 550–600 years, if the depth of cover is increased from 2 inches to 2.5 inches. Two inches of cover was considered to calculate service life to remain conservative. This indicates that service life predictions can vary based on the assumptions made for calculations or real-world circumstances.

Table 5. Service life prediction based on Fick’s second law.

Mix	Diffusion Coefficient	Formation Factor	Predicted Service Life	Corrosion Rate (m/year)
M	$2.90 \times 10^{-14} \text{ m}^2/\text{s}$	65,510	376	$0.92 \times 10^{-6} \text{ m/year}$
M-0.2SF	$3.16 \times 10^{-14} \text{ m}^2/\text{s}$	60,048	345	$1.0 \times 10^{-6} \text{ m/year}$
M-RGP	$2.85 \times 10^{-14} \text{ m}^2/\text{s}$	66,693	383	$0.90 \times 10^{-6} \text{ m/year}$
M-0.3GGBS	$7.43 \times 10^{-14} \text{ m}^2/\text{s}$	25,577	147 *	$2.35 \times 10^{-6} \text{ m/year}$

* Since SR is less than other UHPCs after 28 days, which eventually becomes 1000 $\text{k}\Omega \cdot \text{cm}$.

The results presented here are similar to those in previous research [68], with the primary difference being the aggregate-to-cement ratios. This study has shown slightly higher electrical surface resistivity over the same period.

5.2. Service Life from Freeze–Thaw Resistance

The service life of UHPC can be calculated based on freeze–thaw damage using the following equation:

$$\text{Service life} = \frac{F\text{--}T \text{ cycles required to deteriorate used UHPC}}{F\text{--}T \text{ cycles per year}} \quad (14)$$

If $F\text{--}T$ cycles were required to deteriorate UHPC = 3000 and $F\text{--}T$ cycles per year = 15, then service life would be calculated as follows: Service life = $3000/15 = 200$ years. However, UHPC did not deteriorate even after 600 $F\text{--}T$ cycles. Since it was not tested long enough to deteriorate, it is not possible to find the number of years that UHPC can sustain $F\text{--}T$ damage.

5.3. Techno-Economic Analysis (TEA)

Techno-economic analysis evaluates the technical and economic performance of an emerging technology or product and compares it to an existing one. This analysis is required by the department of energy (DOE) to ensure the sustainability of new technology. In this context, UHPC is the emerging technology, which can be compared to traditional normal or high-strength concrete.

Let us consider a simple double-span bridge, 60 feet in length, with two thermal joints and a bridge deck cross-section, as shown in **Figure 17** below, taken from AASHTO LRFD code [69].

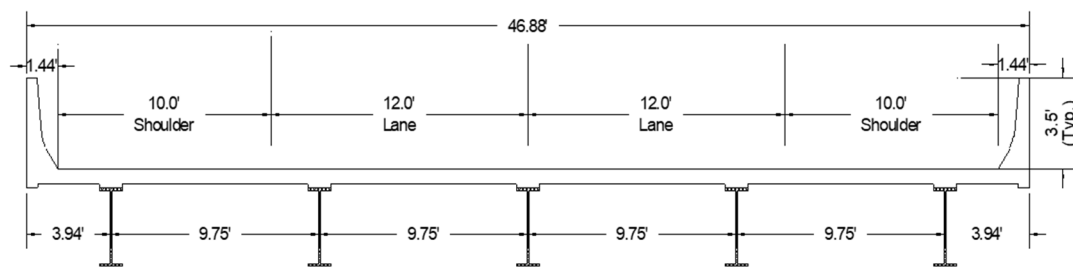


Figure 17. Two-lane bridge adopted from AASHTO LRFD code [68].

Area of bridge deck cross-section: 4817.25 in².

Volume of concrete required for a 60-foot-long bridge: 3,468,420 in³ = 56.84 m³ ~ 57 m³.

Assuming 1% steel reinforcement, the total amount of concrete required for the bridge deck would be 56.4 m³. The cost and *CFP* values are taken from the authors' previous works [33,68], and the embodied energy has been calculated based on available data for the embodied energy of each material constituent [70–73].

The service life of UHPC-built infrastructure is about 350 years, as calculated in Section 5.1, whereas a normal-strength concrete bridge deck has a design life of 75 years. This means that a normal-strength concrete bridge deck would need to be constructed five times over 350 years. However, the value of money decreases over time due to inflation. Historical data show an average inflation rate in the US of about 3.8% per year over the past 60 years, as per US Bureau of Labor Statistics [74]. Therefore, an investment of \$1 today would require \$16 after 75 years.

Table 6 shows the cost, embodied *CFP*, and embodied energy per cubic meter of concrete and for a deck construction. For normal-strength concrete, the deck needs to be reconstructed five times over 375 years. The table clearly shows that the cumulative cost, embodied *CFP*, and embodied energy of normal-strength concrete over five reconstructions are higher than for one-time construction with UHPC. This calculation does not even account for inflation. If inflation is considered, the cost would increase exponentially over time: 16 times in 75 years, 270 times in 150 years, 4410 times in 225 years, 72,310 times in 300 years, and 1,185,789 times in 375 years.

Table 6. Summary of techno-economic analysis of UHPC vs. normal concrete over long term.

	UHPC Mixes						Normal Concrete		
	UHPC-FA		UHPC-RGP		UHPC-GGBS		per m ³	Deck	Five Time
	per m ³	Deck	per m ³	Deck	per m ³	Deck			
Cost (US\$)	933	52,620	973	54,875	934	52,706	250	14,100	70,500
CFP (kg)	700	39,480	743	41,905	610	34,422	408	23,010	115,055
Energy (MJ)	6416	362,055	6652	375,372	5795	327,012	3100	174,840	699,360

This analysis clearly demonstrates that constructing a bridge deck using UHPC is far superior in terms of resource savings—material, cost, *CFP*, energy (fuel), and time. Additionally, traditional bridge joints, typically expected to last 20 years, often last only about 10 years, and in worst-case scenarios, only 5–8 years [75]. This results in frequent major maintenance issues, as frequently as every 10 years. In contrast, a UHPC link slab

could last for 350 years without major maintenance [68]. States like New York, New Jersey, Maryland, Pennsylvania, and Delaware have already started using UHPC link slabs as an alternative to traditional joints [75–79].

5.4. Durability Assessment Factor (κ)

The durability assessment factor (κ) has been introduced to integrate three durability properties investigated in this research: freeze–thaw resistance, resistance to chloride ingress, and shrinkage. This factor provides a comprehensive measure of durability, where higher RDME and resistivity values are favorable, but higher shrinkage values are not. Therefore, X and Y are in the numerator, while Z is in the denominator. To calculate X, Y, and Z in **Equations (16)–(18)**, the numerator values were set to 100 RDME, 254 k Ω ·cm, and 0.13 μ m. An RDME value of 100 indicates no deterioration compared to the original sample, 254 k Ω ·cm is the negligible ion penetration threshold, and 0.13 μ m is the shrinkage criteria set by most of the DOT.

The durability factors are multiplied because they interact and influence one another. For example, if UHPC demonstrates high freeze–thaw resistance, it can generally be expected to exhibit durability in terms of permeability, chloride ingress, and gas diffusion. This is due to the dense matrix structure, which limits the ingress of water and ions. The proposed equation reflects these interdependencies. The proposed equation is

κ = probability of durability in terms of (F–T, chloride ingress, shrinkage)

$$\kappa = \frac{X \times Y}{Z} \quad (15)$$

$$\text{where X is probability of F–T resistance} = - \frac{\text{RDME at 600 F–T cycles}}{100} \quad (16)$$

$$\text{Y is probability of resistance to chloride ingress} = - \frac{28 - \text{day resistivity}}{254 \text{ k}\Omega \times \text{cm}} \quad (17)$$

$$\text{Z is probability of shrinkage} = \frac{28 - \text{day shrinkage}}{0.13 \mu\text{m}} \quad (18)$$

Now, **Table 7** shows the summary of durability assessment based on three durability parameters.

Table 7. Summary durability assessment.

Mix	F–T Resistance (X)	Resistance to Chloride Ingress (Y)	Resistance to Shrinkage (Z)	Durability Assessment Factor (κ)
M	1.05	3.27	0.792	4.33
M-0.2SF	1.02	3.04	0.792	3.92
M-RGP	1.03	3.29	0.792	4.28
M-0.3GGBS	1.03	1.29	0.792	1.69

The overall durability assessment factor is higher for M and M-RGP while, for M-0.2SF, it is 3.92 and, for M-0.3GGBS, it is 1.69. Notably, the resistivity values for M and M-RGP are comparable, while M-0.2SF is slightly lower and M-0.3GGBS has a very low value. Again, M-0.3GGBS due to the inclusion of slag, whose hydration reaction is very slow due to its slow hygroscopicity that resulted in slower increase resistivity values, which further affected the κ . These differences are reflected in the determination of the durability assessment factor. This method can be used to assess the durability properties of the UHPCs.

6. Summary and Conclusions

In this research, durability properties of newly developed, resource-efficient non-proprietary UHPCs were investigated and characterized. A durability assessment factor was introduced to determine the probability of the durability of UHPC structures based on various durability parameters. The service life was determined, and a techno-economic analysis was performed considering the service life and resources utilized. This concept was exemplified by considering an AASHTO-recommended bridge deck. The following conclusions have been drawn from this research:

1. **Freeze-thaw resistance:** All the newly developed non-proprietary UHPCs (with and without fibers) exceeded an RDME of 100%, even after 600 F–T cycles. The increase in dynamic modulus is attributed to the hydration reaction during thawing cycles when the temperature becomes positive. Additionally, an F–T test following ASTM C666 provides a reliable indication of the durability of concrete mixtures.
2. **Permeability:** All newly developed non-proprietary UHPCs (with and without fibers) exceeded the low permeability threshold limit (electrical surface resistivity over 21 k Ω .cm) in less than a month. However, the inclusion of steel fibers affected electrical surface resistivity. Fiber-reinforced UHPCs showed lower electrical surface resistivity values than those without fibers. This suggests that electrical resistivity is a suitable tool for characterizing the durability performance of UHPC matrices without fiber reinforcement. For fiber-reinforced UHPCs, further investigation is recommended to develop empirical formulas and relationships that facilitate durability assessment through electrical resistivity measurements.
3. **Drying shrinkage:** UHPCs without fibers and a W/C ratio of 0.24 showed an average drying shrinkage of 1000 $\mu\epsilon$ in about two months. The same UHPC mixture with fibers of two different fiber lengths (13 mm and 19 mm, both with a diameter of 0.2 mm) showed less than 600 $\mu\epsilon$ of drying shrinkage in the same period. UHPCs with fibers and a W/C ratio of 0.29 exhibited less than 1000 $\mu\epsilon$ within two months.
4. **Absorption:** The absorption of UHPCs was found to be less than 1.4% after 28 days of curing, attributed to the dense microstructure.
5. **Service life:** The estimated service life of these non-proprietary UHPCs is approximately 350 years, based on chloride diffusion theory. This assumes bridge infrastructure in an urban, marine environment in Bridgeport, Connecticut, though factors like climate change may alter projections.
6. **Techno-economic analysis (TEA):** The TEA compared the long-term cost, embodied carbon footprint, and energy usage of UHPC with normal concrete, demonstrating significant resource savings and extended service life.
7. **Durability assessment factor:** The durability assessment factor (κ) was introduced to integrate various durability parameters, providing a comprehensive measure of UHPC durability. These conclusions highlight the potential of UHPC technology to significantly improve the durability and sustainability of concrete structures, encouraging its broader adoption in the construction industry.

7. Recommendation and Future Work

Future studies should conduct life cycle assessments (LCAs) to evaluate the environmental and economic impacts of non-proprietary UHPC, incorporating practical data from DOT or FHWA for greater accuracy. A further exploration of the durability assessment factor (κ) with real-world data is recommended to refine service life predictions. Microstructural investigations, not included in this study, should also be pursued to provide deeper insights into material performance and long-term durability.

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