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Performance Assessment and Design of Ultra-High Performance Concrete (UHPC) Structures Incorporating Life-Cycle Cost and Environmental Impacts

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Abstract:

Ultra-high performance concrete (UHPC) as a novel concrete material is associated with very high strength and low permeability to aggressive environment. There have been many studies focusing on the development of UHPC materials. More studies are needed to implement the knowledge obtained from material level into the structural design and construction level. This paper emphasizes on the structural modeling and performance assessment of bridge girders made of UHPC considering the major improvements in terms of structural performance, durability, environmental impacts, and cost-effectiveness in a long-time interval. Additionally, the effect of the concrete strength increase on the life-cycle environmental impact and cost is assessed on a structural scale. An illustrative example is established to demonstrate the use of UHPC within precast-prestressed girder bridge. It is found that the use of UHPC can result in a significant reduction of concrete volume and CO_2 emissions compared with conventional bridge with the same span length. Additionally, the life-cycle cost and equivalent annual cost associated with these two bridges are compared. This study aims to aid the development and adaptation of novel materials within civil engineering to make optimal use of the favorable material properties.

Keywords: UHPC; Life-cycle cost; Structural reliability and durability; Environmental impacts; CO_2 emissions; Equivalent annual cost.

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1. Introduction

Nowadays, the engineering community, along with the society, is realizing the importance of sustainability-oriented infrastructure systems. Concrete, as a widely used construction material, represents a significant worldwide environmental impacts and cement product is responsible for 7% of the total CO_2 emissions worldwide. Thus, incorporating the sustainability concept into the structural design procedure is of great importance and still remains as a challenging task to the engineers. Sustainable construction and design approaches are critically needed. The concrete industry has devoted effects to reduce the greenhouse gas emissions. For instance, the alternative fuels and substitutions of clinker material by mineral additions were adopted to reduce CO_2 emissions (Haber *et al.* 2010; Wang *et al.* 2018). Another possible solution to help achieve higher infrastructure sustainability is the development and application of new high-performance materials, such as ultra-high performance concrete (UHPC) to reduce the concrete volume used for the infrastructures. However, current construction practice, and design codes and standards fail to permit the full application of the emerging materials. The challenges for the wide use of these materials are generally due to the lack of quantitative tools to evaluate the structural performance, sustainability, and environmental impacts in a long-term range. The relevant research on the application of UHPC within bridges in a life-cycle context is conducted in this paper.

Basically, UHPC, as a new generation of fiber reinforced cementitious material, is composed of cement, fine silica sand, superplasticizer, water, and steel fibers, etc. The development of UHPC represents a significant innovation in the concrete industry and can overcome some shortcomings of normal concrete in terms of strength and permeability to chlorides. Generally, UHPC is associated with extremely high mechanical and durability properties and has the potential to tackle challenges with respect to the load capacity, durability, sustainability, and environmental impacts of concrete structures. For instance, the reinforced concrete structures are easily cracked under service loads, which can cause the ingress of water and chlorides and lead to corrosion of steel reinforcement. Furthermore, the corrosion of steel rebar can lead to spalling of concrete cover and even collapse of structures. UHPC has the potential to address these issues. Thus, the construction of new structures using UHPC can improve the performance and extend the service life, providing a green solution for the development of the next generation of civil infrastructures.

The recent studies on the UHPC on basis of material science aim to produce the materials with high mechanical and durability properties. The application of UHPC in structural design and construction areas has lagged behind the progress on the material level. The knowledge obtained at the UHPC material level should be transferred into the structural engineering and design process. This paper aims to link the behavior of the material to the corresponding structural performance level. UHPC has been adopted for some structures. For instance, a number of bridges have been designed using UHPC in Europe, United States, and China. The first UHPC simply supported bridge with two spans was designed in France and opened to traffic in 2002. Later, a pedestrian bridge in Seoul, Korea, that spans 120 m was constructed. In 2006, the first UHPC bridge in US was built in Wapello County, Iowa. Subsequently, the *pi* girders using UHPC were used in Jakway Park Bridge in Iowa. The first UHPC footbridge, Beichen delta bridge, was constructed in Changsha, China, in 2016. As most of the current structural design codes are related to the concrete with the maximum compression strength of 80 - 85 MPa, which is much lower than the strength of UHPC, the comprehensive design and assessment guidance of UHPC structures is urgently needed in practice. Though there exist some design guidelines for UHPC structures, such as French Interim Recommendation (AFCE 2002) and guidance developed by the Japan Society of Civil Engineering (JSCE 2006), they are not well developed and future modifications are needed. In order to aid the full implement of UHPC within civil infrastructures, the reliability-informed evaluation procedures should be well established and is assessed in this paper.

To aid the development of novel materials within civil engineering, it should ensure that the structures using novel material (e.g., UHPC) should have the least impact on our environment and help to minimize construction and especially maintenance costs in a long term (Müller *et al.* 2014). The production of UHPC is usually associated with high CO_2 emissions and could have an adverse consequence on the environment. Then, life-cycle assessment (LCA) should be incorporated within the evaluation process. LCA is a comprehensive and standardized approach for quantifying life-cycle cost, resource consumption, and environmental impacts of an asset, product, among others. There exist some studies focusing on the UHPC structural performance assessment. Steinberg (2009) examined three analytical approaches to evaluate the ultimate flexural strength of UHPC girders; Almansour and Lounis (2010) presented an initial design and construction approach for the UHPC girder; Van den Heede and De Belie (2012) investigated the structural performance and environmental impact of traditional and novel concrete materials; Gunes *et al.* (2012) presented a two-phase model to investigate the behavior of UHPC and was implemented within a preliminary design case. However, all these studies were focused on the initial structural performance assessment without considering the life-cycle performance and environmental impacts (e.g., CO_2 emissions). In this paper, the long-term benefit of using UHPC within structures is assessed considering a decrease in the amount of concrete used, decrease in maintenance and repair costs, and a longer projected lifespan. In detail, the presented approach considers the interaction between materials and structure, and link them with the life-cycle model to assess the design of infrastructures using novel materials.

In this paper, the life-cycle cost and environmental impact associated with UHPC and normal concrete bridges are assessed considering not only production, design, and construction phases, but also the operation, maintenance, failure, as well as demolition phases. The benefit associated with UHPC structures in terms of structural efficiency, durability, and cost-effective in a life-cycle context is investigated and compared with normal concrete structures. The aim of this paper is to provide evidence that allows the owner and the constructor to make decisions regarding the selection of novel materials for the civil infrastructures that are associated with minimum environmental impact while on the other hand with maximum performance and durability. The rest of this paper is structured as follows: an overview of UHPC material properties, structural reliability, and durability assessment is presented in Section 2. Section 3 discusses the need and methodology for the life-cycle assessment and equivalent annual cost, and environmental impacts assessment (e.g., CO_2 emissions) of UHPC structures. The description of a case study of bridges using normal concrete and UHPC is outlined in Section 4. Finally, section 5 provides the discussion and conclusions of this study.

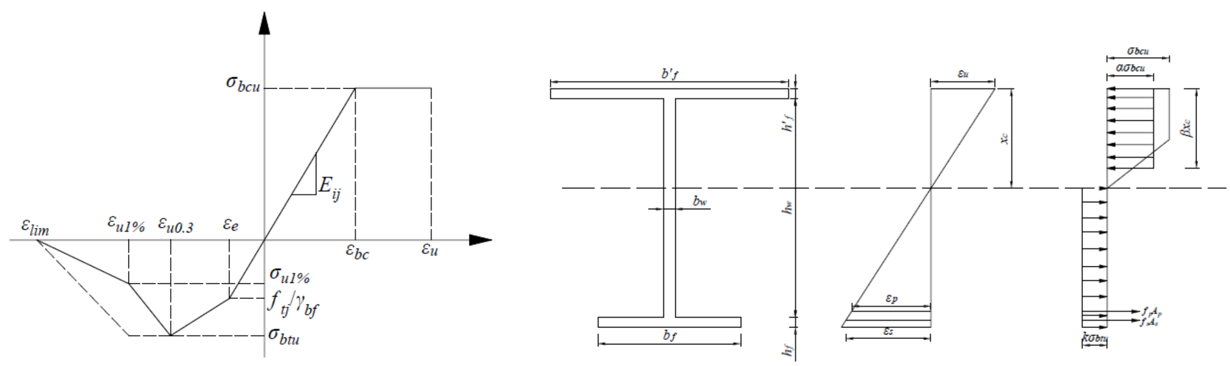
2. UHPC Material and Structural Reliability Assessment

2.1 Mechanical properties and durability of UHPC

UHPC is a class of novel cementitious composite materials that is associated with superior mechanical and durability properties than those of normal concrete (Thomas and Sorensen 2017). UHPC consists mostly of the same constituents as the normal concrete, such as cement, silica fume, water, and quartz sand. However, it also contains finely ground quartz, steel fibers, and superplasticizer. Thus, UHPC tends to be a very low water-to-cementitious ratio composite material with discontinuous fiber reinforcement and little or no coarse aggregate. Due to the low water-to-cementitious ratio, high performance plasticizers are needed to guarantee the workability of fresh UHPC. Due to the existence of the steel fibers, UHPC is associated with a high tensile strength and ductility, allowing the concrete to resist stresses after initial crack (Shafieifar *et al.* 2017). Additionally, the dense nature of the UHPC matrix decreases the porosity of concrete, thus its durability is much better than the normal and high-performance concrete (HPC).

Generally, the mechanical properties of UHPC include compressive strength greater than 150 MPa and sustained post-cracking tensile strength greater than 5 MPa (Graybeal 2011). The tensile strength of UHPC depends on two main factors: the volume of fibers in cross section and fiber orientation. Additionally, due to its dense particle packing and steel fiber, the UHPC exhibits high flexural strength properties. Under given mixture design and curing regime, flexural strength values can be up to 48 MPa (Perry and Zakariasen 2004). Typically, the relationship of stress–strain of UHPC under compression is a linear elastic portion up to 80 – 90 % of the maximum stress value (Graybeal 2006). The stress-strain curve of UHPC obtained from French design recommendations (AFGC 2002) is shown in Figure 1(a) for illustrative purpose. Given more detailed information from the comprehensive lab tests, the model can be updated and easily incorporated.

In Figure 1(a), σ_{bcu} is the ultimate compressive strength; ε_u is the maximum compressive strain; ε_{bc} is the maximum elastic compressive strain; E_{ij} is the Young's modulus; σ_{btu} is the ultimate tension strength; f_{tj} is the limit stress of elastic under tension; γ_{bf} is the partial safety factor; $\sigma_{u1\%}$ is the post-cracking stress corresponding to a crack width of 0.01H; H is the depth of the tested prism with dimensions complying with the structural dimensions; ε_e is the maximum elastic strain under tension; $\varepsilon_{u0.3}$ is the tension strain at σ_{btu}; $\varepsilon_{u1\%}$ is the equivalent tension strain corresponding to a crack width of 0.01H; and ε_{lim} is the ultimate tension strain. With respect to Figure 1(b), B_f' is the effective width of compressive flange; h_f' is the height of compressive flange; b_w is the width of web; h_w is the height of web; b_f is the effective width of tension flange; h_f is the height of tension flange; x_c is the distance from the top fiber of compressive flange to the centroid of the section; ε_s is the tension strain of the reinforcement; ε_p is the tension strain of prestressed tendon; α is the equivalent factor for compressive stress of UHPC in compressive region; β is the equivalent factor for height of compressive region; k is the equivalent factor for tension stress of UHPC in tension region; f_p is the tension strength of prestressed tendon; A_p is the area of prestressed tendon; f_s is the tension strength of reinforcement bar; and A_s is the area of reinforcement bar in tension region



(a) (b)

Figure 1. (a) Stress-strain relationship of UHPC and (b) stress and forces within the UHPC cross-section

UHPC has exhibited high performance on the bond strength to rebar and fibers (Chan and Chu 2004; Alkaysi and El-Tawil 2017). Additionally, UHPC has approved to have high durability properties due to a significant reduction of the volume and sizes of pores. The average pore size in UHPC was found to be less than 5 *nm* and it has very low water absorption capacity. Due to the improved permeability and porosity of UHPC, it is associated with much better resistance to freezing-thawing cycles (Graybeal 2006). For instance, Acker and Behloul (2004) stated that after 300 freeze-thaw cycles, UHPC with no degradation. The water permeability coefficient of UHPC was reported to be about 0.0005 (Heinz and Ludwig 2004), which is significantly smaller than that of the normal concrete. Another property with respect to durability of UHPC is the chloride icons penetration. It was found that the chloride diffusion coefficient of UHPC is significantly lower than that associated with normal concrete. Additionally, accelerated tests were conducted to evaluate the chloride penetration depth within UHPC specimens. Steel fibers in UHPC did not cause any electrical short circuiting during the rapid chloride ion penetrability test (Ahlborn *et al.* 2008). Additionally, the corrosion rate of reinforcement in UHPC was tested to be approximately 0.01 μm/year, which is much lower than the threshold value of 1 μm/year (Roux *et al.* 1996). Ahlborn *et al.* (2008) found that the UHPC could be considered to have a negligible chloride penetration. Thus, the UHPC with highly density and reduced pore volume can resist aggressive chemicals and water from entering the cementitious matrix, thus can prevent the deleterious solutions from penetrating into the concrete (AFGC 2002; Schmidt *et al.* 2003). Overall, the durability properties of UHPC, obtained from the permeability tests, freeze-thaw tests, abrasion tests, etc., are significantly better than those of normal concrete and HPC (Abbs *et al.* 2016). One possible disadvantage with respect to UHPC is its resistance to fire and elevated temperatures. The relevant advantages and disadvantages of UHPC are indicated in Table 1.

Table 1. Advantages and disadvantages of UHPC

Advantages	Disadvantages
• UHPC is associated with very high compressive strength and can gain compressive strength rapidly; • Tensile strength of UHPC, both before and after tensile cracking, is significantly higher than that occurs in the normal concrete; • UHPC can exhibit ultra-high ductility, e.g., very high ultimate compressive and tensile strain; • UHPC is with a better workability considering its high fluidity; • UHPC displays excellent durability properties that are significantly beyond those associated with normal concrete. Additionally, UHPC is found to be innocuous to alkali-silica reaction; • UHPC performs exceptionally well by considering the toughness indices;	• UHPC is associated with high cost, including cost for the constitutive materials and curing; • UHPC could result in a high autogenous shrinkage considering its relative low water-binder ratio; and • A rigorous requirement is needed for the curing environment if high early-stage strength and lower later-stage shrinkage are needed.

• The cracked UHPC split cylinders usually do not result in a noticeable decrease in the peak tensile load-carrying capacity under an aggressive environment; and • UHPC is with much lower drying shrinkage and creep after steam treatment.	

2.2 Reliability analysis

A reliability analysis is needed to investigate the performance of UHPC bridges and then aid the design of UHPC bridges. Structural reliability analysis takes into account the uncertainties and variability and can be defined as the probability that a component or a system will adequately perform its specified purpose under particular conditions. For instance, if R and S represent the resistance and the demand, respectively, the probability density functions (PDFs) f_R and f_S, characterizing these respective random variables can be established as indicated in Figure 2(a). The structural performance highly relies on the quality and related variability of both the demand S and the resistance R. The probability that S will not exceed R, $P(R > S)$, represents the reliability. As a general case, probability of failure p_F can be expressed in terms of joint PDF of the random variables R and S, $f_{R,S}$, as follows:

$$p_F = \int_0^{\infty} \left(\int_0^{s} f_{R,S} dr \right) ds \tag{1}$$

Furthermore, the reliability index can be expressed as:

$$\beta = \Phi^{-1}(1 - p_F) \tag{2}$$

where $\Phi^{-1}(\cdot)$ is the inverse of the standard normal cumulative distribution function (CDF).

In addition to evaluate the probability of structural failure, it is also possible to consider various limit states that affect infrastructure systems such as serviceability. The serviceability limit state is usually established considering the deflection and crack size of the structure under given design load. The ultimate limit state should also be considered. The flexural ultimate limit state is emphasized herein. In order to compute the ultimate capacity, the distribution of the strain along the cross section should be determined. The multilinear stress-strain curve can be used in this paper to determine the stress and strain relationship of UHPC (Steinberg 2009). The stress distribution along the plane is shown in Figure 1(b) considering the stress-stain relationship. More detailed information regarding the idealized stresses and forces within a cross-section can be found in Steinberg (2009). Then, the location of neutral axis is computed based on the force equilibrium. Subsequently, capacity is calculated based on the force and moment arm.

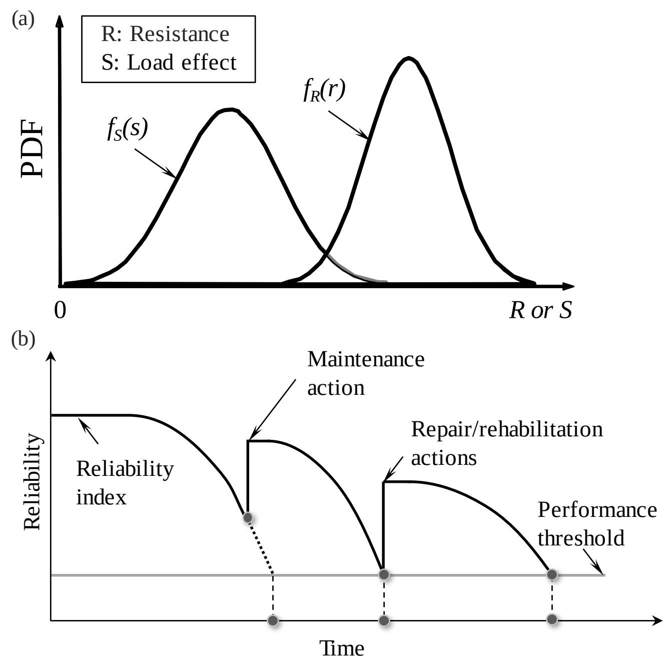


Figure 2. (a) Probability density function of *R* and *S* and (b) the timing and effect of maintenance actions on the performance of a deteriorating structure

The ultimate flexural failure associated with the investigated structure is introduced herein. Given the load effect and capacity, the performance function associated with structural failure under bending moment can be identified as

$$g = M_u - (M_{DL} + \eta_{df} M_{LL}) \tag{3}$$

where $M_u(t)$ is flexural resistance (capacity); M_{DL} is dead load moment; η_{df} is dynamic load amplification factor; and M_{LL} is live load moment. Given the uniformly-distributed load associated with self-weight of the girder, the dead load effect M_{DL} can be computed. The live-load flexural moment can be computed given the axel spacing and weight distribution of the specific truck. Given the worse locations of the live load on the longitudinal and transverse directions, the given flexural moment of live load can be determined. Additionally, the uncertainties associated with the geometric parameters and material properties should be considered within the evaluation process. The reliability index can be computed using Eq. (2). A threshold with the prescribed performance indicator should be determined. The reliability index is selected to present the performance level of the structure. When the selected performance indicator reaches the threshold value, the structure would reach its service life. The reliability threshold selected is set as 3.5, corresponding to a probability of failure 2.3×10^{-4}. This estimated reliability β_{target} is also used as the criterion for the evaluation of bridge performance within AASHTO LRFD specifications.

2.3 Structural durability assessment

The structural deterioration is related to many factors, such as inadequate construction materials, bad construction practices, deferred maintenance actions, ignorance of severe environmental, among others. Bridges are directly subjected to environmental attacks and their performance usually deteriorates with time. The common durability issues within the reinforced concrete bridges are corrosion of rebars, sulphate and other chemical attack, alkali aggregate reaction, freezing and thawing damage, and carbonation, among others. Corrosion of reinforcement steel is one of the primary sources of deterioration mechanisms. The poor quality of the constituent material and workmanship could lead to early corrosion of reinforcement, which mainly depends on corrosion initiation time and corrosion rate. Previously, durability issue has not been well considered within bridge design process and has led to severe deterioration of bridges, which has been a serious problem for the bridges worldwide. The process for the durability assessment composes of several steps. First, the environmental scenarios should be determined. Then, the mechanisms of the deterioration scenario are needed to be identified, considering chloride-induced corrosion, carbonation-induced corrosion, mechanical abrasion, salt weathering, surface deterioration, and frost attack, etc. These deterioration scenarios can cause a reduction of the cross-sectional area of concrete and steel, and spalling of concrete cover (Dong *et al.* 2013; Sabatino *et al.* 2015). Given the deteriorating structural performance, maintenance actions should be applied to the structures to either delay the occurrence of reaching a performance threshold or improve the performance once the threshold is reached. The maintenance actions could be categorized into two types: preventive and essential (Kong and Frangopol 2003). Preventive maintenance (PM) can stop or slow down deterioration rate. For instance, the PM includes bridge washing, seal deck joints, sealing concrete, painting steel, *etc*. Essential maintenance (EM) is applied to the deteriorating structures when the performance indicator reaches the prescribed threshold value, which usually has a significant effect on the structural performance and is associated with a relatively higher cost. The EM includes the replacement of the damaged components with new ones. As the UHPC is a relatively new material, the detailed information of the deterioration mechanism of UHPC structures is still limited and more numerical and experimental studies should be conducted to assess their long-term performance. Once these information (e.g., deterioration mechanisms and loading effect) is available, the time-variant performance indicators (e.g., reliability) can be predicted. Subsequently, given the predicted lifetime performance and types of maintenance actions (e.g., PM and EM), the timing and specific type of maintenance actions can be determined to improve structural performance.

As the durability is a critical problem for the bridges using normal concrete, the durability assessment of bridges under chloride ingress is introduced. Chloride ion migration through a concrete by means of capillary absorption, hydrostatic pressure, or diffusion is one of the most problematic durability issues associated with low permeability concretes (Stanish *et al.* 2000). Concrete with high permeability is susceptible to chloride ingress which eventually can lead to corrosion of reinforcing steel. Once chloride ions reach embedded steel, corrosion can take place through an electro-chemical reaction that expands the steel up to 600%. Based on the geometric shape of reinforcement steel after corrosion, reinforcement corrosion can be divided into two categories: one is the uniform corrosion and the other is chloride-induced pitting corrosion. According to Gonzalez *et al.* (1995), the maximum penetration of pitting corrosion is about four to eight times of that associated with uniform corrosion, thus pitting corrosion can lead to a more severe area loss. The diffusion process of chloride ions through concrete surface can be modeled by the Fick's law; then, given the geometric model, the loss of effective cross-sectional area under pitting corrosion can be computed (Val and Melchers 1997). As the prestressed tendon usually consists of strands with several twisted wires, and is placed inside the grouted corrugated ducts, the corrosion mechanism of prestressed tendons is more complex than that of reinforcement. With respect to the prestressed strand, it is usually assumed that the pitting only formed on the outer wires (Darmawan and Stewart 2007). In addition to the loss of net area, the corrosion can affect the strength of reinforcement steel and can also be accounted for within the evaluation process (Vu *et al.* 2009). As these deterioration scenarios can cause a reduction of the cross-sectional area of the reinforcement and prestressed strand, the structural capacity will be reduced accordingly and will reduce the structural performance levels in terms of reliability index. The qualitative figure of structural performance in a life-cycle context considering structural deteriorations is shown in Figure 2(b).

Previous studies demonstrated that UHPC exhibited almost no permeability and was not susceptible to chloride ingress. Performing rapid chloride permeability tests, it showed that UHPC was capable of achieving permeability values less than 100 coulombs for both air-cured and steam-cured specimens (Ahlborn *et al*. 2008). Materials with coulomb values less than 100 are generally with negligible chloride ion penetration. Recently, it has been recognized that UHPC provides a possible solution to structural crack by smearing one or several dominant cracks into many distributed microcracks. Thus, in this way, the UHPC can improve the structural durability significantly. The investigated bridge in this paper has been open for service and can provide useful information regarding the maintenance actions for the UHPC bridges in the life-cycle. Once these information is available, it could be easily incorporated within this study to aid the design, assessment, and management of UHPC structures.

3. Life-Cycle Assessment and Environmental Impact

Life-cycle assessment (LCA) is defined as the evaluation of a system throughout its life cycle from cradle to grave (Kim and Frangopol 2012). Generally, higher initial cost with a better quality can result in a lower operation and maintenance cost. In this way, the structural performance should be investigated in a life-cycle context. LCA is a tool for systematically analyzing structural performance of products or processes over their entire life cycle, including raw material extraction, manufacturing, construction, inspection, maintenance, repair, rehabilitation, and end-of-life disposal and recycling. The flowchart of the life-cycle framework is shown in Figure 3.

3.1 Environmental impact assessment

A typical life-cycle environmental impact assessment considers different indicators, such as global warming potential, acidification of land and water sources, human toxicity, formation of tropospheric ozone, and depletion of nonrenewable recourses, among others. The global warming potential is chosen as the environmental indicator herein. The cumulative environmental impact in terms of CO_2 emissions of the infrastructure over its service life is expressed as global warming potential.

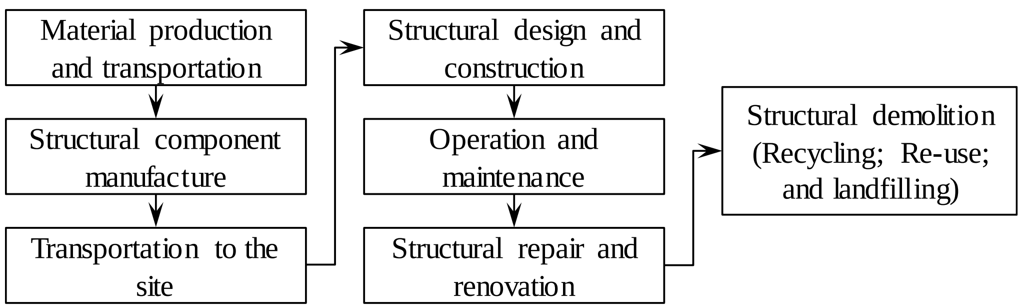


Figure 3. Flowchart of the life-cycle assessment framework

The first phase considered is the material manufacture phase considering activities of raw materials extraction, sub-material transportation, etc. Generally, the environmental impact in terms of CO_2 emissions can be calculated by multiplying the impact associated with each raw material considering the amount of raw materials used (García-Segura *et al.* 2014). The total enviomental impact, CO_2 emissions, of the investigated strcture is the sum of the unit CO_2 emission weighted by the volume of all the materials of the bridge. Thus, the CO_2 emissions during the manufacture stage considering all the materials used in a bridge are computed as (Bouhaya *et al.* 2009)

$$EN_{manu} = \sum_{i=1}^{n_m} en_i \cdot V_i \tag{4}$$

where n_m is the number of the materials that used within the investigated structure; en_i is the CO_2 emissions associated with manufacture phase of a unit of material i (kg-CO_2/m^3); and V_i is the volume assoicated with materail i, e.g., concrete, steel, rebar, etc. The unit CO_2 emission of different materials is related with many factors. Different industrial variabilities can affect the value of the CO_2 emissions during the production process.

The unit CO_2 emissions associated with concrete $EN_{c,manu}$ are introduced. The CO_2 emissions associated with manufacture phase of a unit of concrete (kg-CO_2/m^3), $EN_{c,manu}$ can be computed as

$$EN_{c,manu} = \sum_{i=1}^{n_{cm}} ma_i \cdot cefm_i \tag{5}$$

where ma_i is the amount of the materials used in the concrete (kg/m^3); $cefm_i$ is the CO_2 emission factor associated with manufacture of matrial i (kg-CO_2/kg); and n_{cm} is the number of mixes within the concrete. For instance, UHPC contains steel fibers, silica fume, cement, fine aggregates, a water reducing agent, and water. Given the CO_2 emission associated with different composers within UHPC, the total CO_2 emissions with production phase of UHPC can be obtained.

The following phase considered within environmental impacts assessment is transportation phase, which depends on the distance and manner of transportation. Specifically, the CO_2 emissions associated with transportation of the structural material and/or components from the factory to the site are considered. Basically, the steel is transported over a relatively long distance than the normal concrete and the transportation distance of UHPC materials and components is generally larger than those of normal concrete. The CO_2 emissions with respect to transportation phase, EN_{trans} can be computed as

$$EN_{trans} = \sum_{i=1}^{n_m} bma_i \cdot V_i \frac{d_i}{fe} \cdot ceft \tag{6}$$

where bwa_i is the unit weight of material i kg/m^3 used within the bridge; d_i is the transportation distance of material i (km); fe is the fuel efficiency (km/L); and $ceft$ is the CO_2 emission factor associated with transportation of fuel consumption (CO_2 kg/L-kg).

Another phase considered is construction, which is related with the construction methodology. For instance, different construction techniques use different construction machines and, thus, result in different energy efficiency and environmental impacts. The environmental impact within this phase mainly accounts for the electricity consumption. For instance, for a bridge weight being close to 100 *t*, a 500 *kW* power crane is necessary to put the bridge deck in place. The UHPC components are light enough to be constructed efficiently, which can save tremendous construction time compared with the conventional bridge. Generally, the emissions during construction are almost negligible compared to those during material production.

The subsequent phase considered within the assessment process is maintenance, repair, and rehabilitation (MR&R) phase. Regarding the MR&R interventions, they can be categorized into two types: Preventive and essential maintenance actions. Preventive maintenance can stop or slow down the deterioration rate of the structure, which can be applied at regular or irregular time intervals. Preventive maintenance includes replacing small parts, patching concrete, repairing cracks, changing lubricants, and cleaning and painting exposed parts, among others. The inspections include the regularly scheduled and special inspection events to assess the condition of the bridge under deterioration, extreme events, etc. and can be regarded as preventive maintenance actions. The essential maintenance is based on the performance of the structure, which should be applied when the performance indicator reaches the pre-defined target value. The essential maintenance action can have a significant effect on structural performance. As the essential maintenance action can cause bridge partial or total closure, it could result in much higher user costs compared with preventive maintenance actions. The essential maintenance generally includes replacing a bearing, resurfacing a deck, or modifying a girder of bridge. The scheduling of MR&R depends on the investigated limit state, loading scenarios, budget planning, and the consequences of failure. The number of MR&R actions can be determined considering the deterioration behavior of the bridge. For instance, given the chloride concentration is larger than the prescribed threshold value, the preventive repair action is applied to the damaged component/structures. The environmental impacts in terms of CO_2 emissions are related with the characteristics of the repair technique. Herein, the CO_2 emissions of MR&R actions of the bridge are usually assessed as a fraction of the initial value of a bridge. Accordingly, the CO_2 emissions associated with the MR&R action within a life-cycle context are

$$E_{main} = \sum_{i=1}^{n} E_{env,ini} \cdot r_{mi} \tag{7}$$

where n is the number of maintenance actions within life-cycle; $E_{env,ini}$ is the initial CO_2 emissions associated with production, transport, and construction; and r_{mi} is the ratio between the CO_2 emission of maintenance action i and the CO_2 emissions of $E_{env,ini}$. As the MR&R activities are associated with a traffic closure, this can cause environmental impacts for the traffic to detour. Given more information, the environmental impact associated with bridge downtime under MR&R can also be assessed.

The end of life phase includes the demolition of structure, material treatment, recycling process, and the associated transportation. The material treatment and recycling have the potential to benefit our environment, considering the recycling and reused materials. For instance, the normal concrete is usually crushed and used as lower quality aggregates for backfills or road. The dumped UHPC can be suitable for the lower strength concrete. The steel can also be recycled. Under a specific scenario, the CO_2 emissions could be transferred in the monetary unit, which depends on many parameters (e.g., science, economics, climate change issues) and may vary for different sources. Once the unit monetary cost of the CO_2 is available, the monetary value of CO_2 emissions during the investigated time span can be easily obtained.

3.2 Life-cycle cost analysis and equivalent annual cost

Life-cycle cost analysis (LCCA), is a decision support technique that enables comparison of LCCs of alternatives that may all fulfill a certain function but differ substantially in a life-cycle context (Safi *et al.* 2012). As the same procedure developed previously for the environmental impacts assessment, LCC contains the costs associated with not only design and construction phase, but also the operation, maintenance, failure, as well as demolition. The characteristic life cycle of a concrete structure is shown in Figure 3. Based on the life-cycle stages of a structure, the total cost during the lifetime of a bridge can be expressed as (Frangopol *et al.* 2017)

$$LCC_{NPV} = C_{cons} + \sum_{j=1}^{n_{ins}} \frac{C_{ins,j}(t_j)}{(1+r)^{t_j}} + \sum_{k=1}^{n_{mt}} \frac{C_{mt,k}(t_k)}{(1+r)^{t_k}} + \sum_{i=1}^{n_{fe}} \frac{C_{fc,i}(t_i)}{(1+r)^{t_i}} + \frac{C_{dm}}{(1+r)^T} - \frac{R_v}{(1+r)^T} \qquad (8)$$

where r is the monetary discount rate and is used to transfer all the future money into the present value; C_{cons} is the initial construction cost that includes material and labor cost, *etc*; C_{ins} is the inspection cost; C_{mt} is the maintenance cost in terms of the direct and indirect cost of maintenance actions; C_{fl} is the failure cost in terms of the direct and indirect cost caused by structural failure; C_{dm} is the demolition cost, including the costs demolition, landfill and recycling; R_v is the residual monetary value of the structure at the end of its designed service life given it does not fail; T is the investigated life-cycle (e.g., designed service life); and n_{ins}, n_{mt}, and n_{fe} are the number of inspection, maintenance, and failure events, respectively, during the investigated life-cycle. Both direct and indirect cost can be considered within the cost assessment process. Generally, direct cost could refer to the money spent directly on engineering activities, while the indirect cost is related with the deficiency or loss of structural functionality. For instance, bridge failure can paralyze the traffic network, induce time cost to the commuters who travel through it, and economic loss to regional industry. These costs are considered as indirect costs.

In order to compute the life-cycle cost, the initial construction cost should be computed firstly. Generally, the C_{cons} consists of the labor cost, engineering equipment cost and material cost associated with all the materials within the investigated structure. Thus, C_{cons} can be computed as

$$C_{cons} = \sum_{i}^{n_m} c_{m,i} \cdot V_i \cdot \rho_i \tag{9}$$

where c_{mi} is the unit cost per ton of material i (e.g., steel, normal concrete, UHPC); V_i is the total volume of material i; ρ_i is the density of material i; and n_m is the number of materials within a structure. The initial material cost of UHPC is much higher than that of normal and HPC. The price of UHPC in rarely used region can be relatively higher than that in the area of commonly used. The unit price of UHPC can be approximately ten times of that of the normal concrete (Ngo 2016). The wide application of UHPC can result in a price reduction of UHPC. Also, the price of UHPC depends on the application of different technologies, such as alternative mixes.

Regarding the normal bridges, the relevant inspection and maintenance cost can be obtained based on the historical data of the same type of bridges. With respect to novel structure, there are no available data on the relevant maintenance actions and sound engineering prediction can be made based on the professional judgement. Generally, the inspection and maintenance cost is proportional to the construction cost. During the life-cycle of the bridge, it can also subject to extreme events, which can cause the bridge failure. Given the occurrence rate and intensity of the extreme events, the relevant loss in a life-cycle context can be obtained (Dong and Frangopol 2016). At the end of the investigated service life of a bridge, it still has a useful life and the residual value should be considered. The residual value of the structure can be taken as the percentage of the initial construction cost. This is related with the types of the materials used within the structure. The demolishing cost can be as a proportion to the initial construction value.

Eq. (8) mainly aims to compute the life-cycle cost of bridges with the same lifespan under given investigated scenarios (e.g., monetary discount rate, construction cost, life-span). The life-cycle cost of bridges with different lifespans (e.g., design service life of normal bridge and UHPC bridge) can not be assessed rationally by using this equation (Jones and Smith 1982). In other words, the cost-effective bridge may not be the alternative that is associated with a lower maintenance and rehabilitation cost and longer life-span, as it can result in a relatively larger life-cycle cost compared with the bridge with a smaller life-span. In order to address this issue, the equivalent annual cost should be adopted as another performance indicator for the comparison of the assets with different service lives (Jones and Smith 1982; Kauffmann *et al.* 2012). The equivalent annual cost is the annual cost of operating and maintaining an asset over its entire life and is widely used to compare the cost-effectiveness of various assets that have different service lives. . In this way, the alternative with the lowest equivalent annual cost is the most efficient and cost-benefit options. The equivalent uniform annual costs LCC_{EUAC} can be computed as

$$LCC_{EUAC} = \begin{cases} LCC_{NPV} \dfrac{r \cdot (1+r)^{T}}{(1+r)^{T} - 1}, & r > 0 \\ \dfrac{LCC_{NPV}}{T}, & r = 0 \end{cases} \tag{10}$$

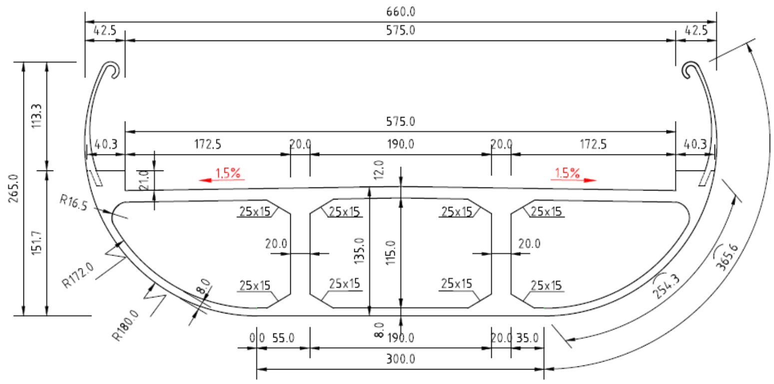
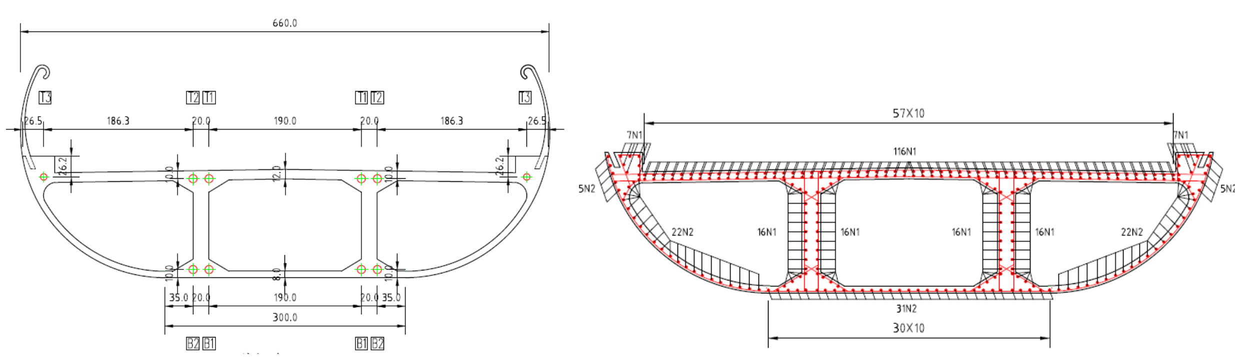


Figure 4. Bridge configuration (dimensions in cm) in the mid-span section of the UHPC bridge

4. Illustrative Example

The presented approach is applied to a UHPC bridge, located in Changsha, China (Wei 2015). The bridge is composed of one girder with two spans. A transverse cross-section of the bridge at the mid-span is shown in Figure 4, with a width of 6.6 m. The reinforcement arrangement (e.g., prestressed tendon and normal reinforcement) in the mid-span is shown in Figure 5. To demonstrate the benefit associated with UHPC bridge, a comparative study is conducted to investigate the performance of two girder bridges using conventional and UHPC. Both bridges are designed to have the same reliability index associated with flexural failure in order to make the performance of the two bridges comparable.



(a) (b)

Figure 5. Reinforcement arrangement: (a) prestressed tendon and (b) normal reinforcement bar, mid-span section (dimensions in cm) of UHPC bridge

4.1 Reliability analysis of UHPC bridge

The flexural failure, as a main failure mode of the reinforced concrete structure, is emphasized in this paper. Given more information, other failure scenarios, such as shear failure, could also be considered within the evaluation process. In order to compute the reliability, the material properties of UHPC under compression and tension should be identified. Given the geometry of the cross-section and stress-strain relationship, the bending moment capacity of the cross-section can be computed. More detailed information can be found in Steinberg (2009). The random variables involved in geometric parameters, initial material properties, and load effects are considered within the assessment process as indicated in Table 2. The post-cracking tensile strength of UHPC depends on many parameters (e.g., distribution and orientation of embedded fibers) and could result in a large scattering response. In this paper, the parameters associated with distribution type of post-cracking tensile strength are based on the experimental results. More experimental studies are needed to investigate the uncertainty associated with the mechanical properties of UHPC. The probability density function of the ultimate bending moment capacity is shown in Figure 6 by using Monte Carlo simulation. Then, given the load effect and capacity, the probability of failure of the cross section under the bending moment can be computed using Eqs. (1) and (2) and the reliability index β of the UHPC girder bridge is computed.

Table 2. Random variables used in the reliability analysis process of UHPC bridge

Variable	Properties	Mean	COV	Distribution type
b'_f(m)	Effective width of top flange	6.6 × 1.0013	0.0081	Normal
h'_f(m)	Height of top flange	0.12 × 1.032	0.1019	Normal
b_w (m)	width of web	0.56 × 1.032	0.1019	Normal
b'_f(m)	Effective width of compressive flange	6.6 × 1.0013	0.0081	Normal
h'_f(m)	Height of compressive flange	0.08 × 1.032	0.1019	Normal
h_0	Effective height of section	1.25 × 1.0124	0.0229	Normal

h_w	Height of section	1.17 × 1.0064	0.0255	Normal
A_s (mm^2)	Area of tension reinforcement bar	1559	0.0350	Normal
A'_s (mm^2)	Area of compressive reinforcement bar	10708	0.0350	Normal
A_p (mm^2)	Area of tension prestressed tendon	9452	0.0350	Normal
A'_p (mm^2)	Area of compressive prestressed tendon	9174	0.0350	Normal
f_c (MPa)	Compressive strength of UHPC	164	0.0734	Normal
f_t (MPa)	Tension strength of UHPC	7.8	0.12	Normal
f_s,f'_s (MPa)	Ultimate strength of reinforcement bar	369	0.0719	Normal
f_p,f'_p (MPa)	Ultimate strength of prestressed tendon	1860 × 1.0387	0.0142	Normal
M_{DL}	Bending moment due to dead load	1.0148 × 10^4	0.0431	Gumbel
η_{df}	Dynamic load amplification factor	1.2	0.067	Normal
M_{VL}	Bending moment due to live load	1.273× 10^4	0.1858	Gumbel

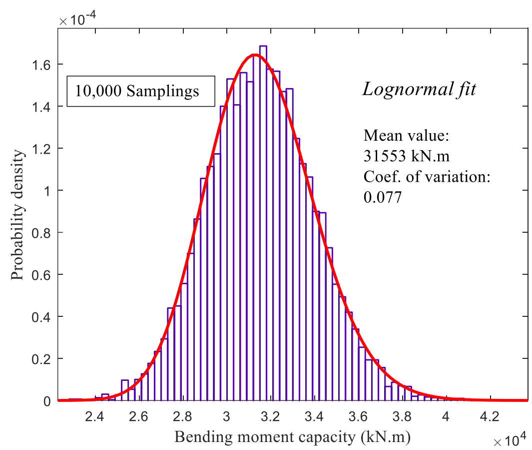


Figure 6. Probability density function of bending moment capacity of mid-span section

In order to compare the performance of structures using UHPC and normal concrete, the girder using the normal concrete is also investigated and these two bridges have the same reliability index. Then, the cross-section of the girder using normal concrete is established based on the relevant bridge design code. The cross section of the bridge using normal concrete is indicated in Figure 7(a) and the reinforcement arrangement is shown in Figure 7(b) accordingly. The area of the materials used in the cross section of the normal and UHPC girder is indicated in Table 3. As indicated, the use of UHPC can result in a significant reduction of the concrete volume. By comparison, by using UHPC, the volume of the concrete can be reduced by 48%. Furthermore, the usage of the reinforcement is also reduced significantly by using UHPC. Accordingly, the uses of rebar and prestressing steel are reduced by 39% and 44%, respectively. Thus, the UHPC results in 51.3% reduction of the total weight of the superstructure. The reduced weight of UHPC superstructure can lead to a considerable reduced size of the substructure.

Table 3. Area of materials used in the section of UHPC and normal bridges

Bridge	Material	Strength grade	Area
UHPC bridge	UHPC	UHPC	2.1511 m^2
	Reinforcement tendons	HRB 335	12267 mm^2
	Prestressed tendons	Ultra-high steel strand (15.2 mm, 7Φ5)	18626 mm^2
Normal bridge	Normal concrete	C50	4.145 m^2
	Reinforcement tendons	HRB 335	20106 mm^2
	Prestressed tendons	Ultra-high steel strand (15.2 mm, 7Φ5)	33360 mm^2

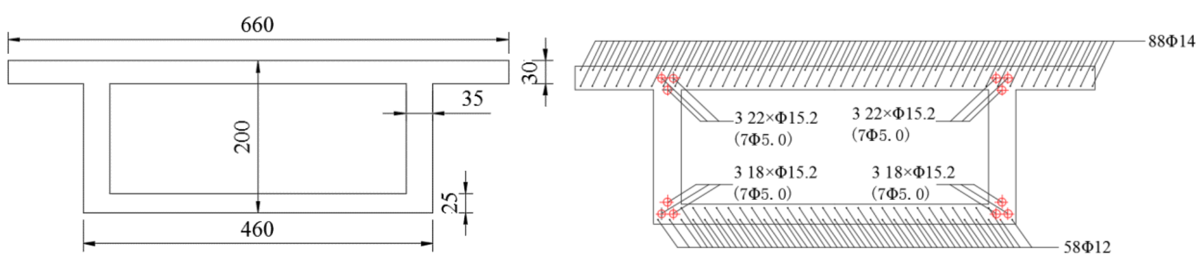


Figure 7. (a) Section of normal concrete bridge (dimensions in cm) and (b) the arrangement of reinforcement within the cross section

4.2 Environmental impacts assessment

The structural performance indicators (e.g., environmental impacts, life-cycle cost) are assessed for the normal bridge and UHPC bridge. The computational flowchart of the performance indicators is shown in Figure 8. In this section, the environmental impacts associated with these two bridges are investigated in a life cycle context. During the production phase of UHPC, a cubic meter of UHPC requires approximately 800 kg of cement, 200 kg of water, 250 kg of silica fume, 750 kg of aggregates, 40 kg of water reduce agent, and 195 kg of steel fibers. The unit CO_2 emissions associated with cement, aggregate, steel fiber, and water reduce agent are 0.865 kg-CO_2/kg, 0.0013 kg-CO_2/kg, 0.94 kg-CO_2/kg, and 0.0184 kg-CO_2/kg, respectively (Bouhaya *et al.* 2009). By using Eq. (5), the CO_2 emissions during the production of one cubic of UHPC are 877 kg-CO_2/m^3 by using the parameters provided in this paper. The CO_2 emissions within the initial manufacture process with respect to the cement and steel fibers are 692 and 183.3 kg-CO_2/m^3, respectively. As indicated, cement contributes approximately 78.9% of the total emissions during the production stage. By considering the total volume of UHPC used within the bridge, the CO_2 emissions associated with UHPC of the investigated bridge are 1.3394×10^5 kg. The unit CO_2 emissions of the reinforced bar and prestressing steel are 3.03 and 5.64 kg-CO_2/kg, respectively and the CO_2 emissions associated with the steel used in the UHPC bridge can be computed accordingly. The total CO_2 emissions of the production of reinforcements within the UHPC bridge are 6.004×10^4 kg.

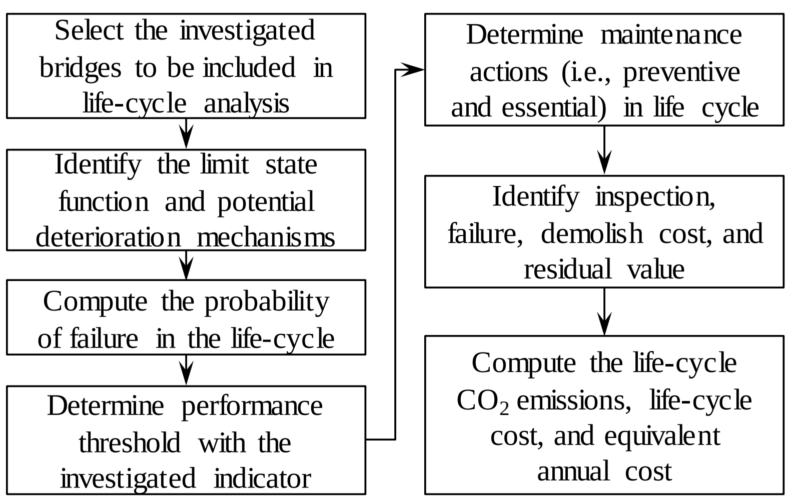


Figure 8. Computational flowchart of structural performance indicators (e.g., life-cycle cost, equivalent annual cost) in a life-cycle context

Similarly, the CO_2 emissions associated with the production of the normal concrete can be computed and the total CO_2 emissions of normal concrete are 348.3 kg-CO_2/m^3 of normal concrete, which is around 39.7% with respect to the UHPC. Thus, during the production process of per cubic concrete, UHPC has a much severe adverse consequence to the environment. By considering the total volume of the normal concrete used within the bridge, the total CO_2 emissions associated with all the concrete used in the normal bridge are 1.025×10^5 kg, which is approximately 73.3% of UHPC bridge. The total CO_2 emissions of the reinforcement used in the normal bridge are 7.5036×10^4 kg. As the volume of the reinforced used in the normal bridge is larger than that of the UHPC bridge, the relevant CO_2 emissions of the reinforcement used in the normal bridge is larger than that of UHPC bridge.

Given the total volume of the concrete and steel used within conventional and UHPC bridges, the total CO_2 emissions of the production phase of the materials used in these bridges are computed and are 1.7754×10^5 kg and 1.94×10^5 kg, respectively. The CO_2 emissions associated with the initial state of the UHPC bridge are higher than that of the normal bridge considering the material production. By using UHPC, the CO_2 emissions of the bridge associated with material production increase approximately 10%.

The CO_2 emissions during the transportation stage of the raw materials to the concrete plant and bridge site are assessed herein. The transportation distance associated with cement, aggregate, steel fiber, and silica fume are 100, 100, 500, and 500 km, respectively. Basically, the distances are determined based on the specificity of the materials. For instance, usually the traveling distance of steel fiber and silica fume is larger than that of the concrete and aggregate. Using Eq. (6), the release of CO_2 from a truck is proportional to the fuel consumption and the ratio *ceft* is taken as 2.7 kg-CO_2/L-ton (0.003/L) for standard fuel and the fuel efficiency *fe* is 0.38 L/km (Bouhaya *et* *al.* 2009). Subsequently, by using Eq. (6), the CO_2 emissions during the transportation phase are 1.882×10^4 kg for the UHPC bridge. Accordingly, the production phase can result in a much larger amount of CO_2 emissions than that associated with transportation. The CO_2 emission associated with the transportation stage of normal bridge is 7.5027×10^4 kg, which is much larger than that of the UHPC bridge.

The CO_2 emissions associated with MR&R of these two bridges are computed, which could have a great effect on the life-cycle assessment process. The investigated life span of the concrete bridge is considered to be 75 years. The type and frequency of these activities of the normal bridge are based on the historic data of similar bridges. For the conventional bridge, the interval for the preventive and essential actions are determined as 10 and 40 years, respectively. The CO_2 emissions of the preventive and essential maintenance actions are regarded as a percentage of that during total of production and transportation phases. The CO_2 emissions ratio with respect to the preventive and essential maintenance actions are deemed as 0.1 and 0.5 of the initial value. As the UHPC can extend the service life of the bridge, the frequency for the MR&R is reduced accordingly. Accordingly, the UHPC structure needs relatively smaller amount of maintenance and repair actions compared with the conventional concrete. The number of the preventive action during the service life of the UHPC is determined as half of that associated with normal bridge. Additionally, as the service life of UHPC bridge is extended to larger than 75 years, no structural essential maintenance action is needed for the UHPC bridge. Thus, the CO_2 emissions of maintenance action during the life-cycle of the normal and UHPC structures are 2.308×10^5 and 7.759×10^4 kg, respectively. Given more information, the relevant parameters could be easily implemented. The CO_2 emissions of these two bridges during the investigated stages in a life-cycle context is shown in Figure 9. The total CO_2 emissions of UHPC and normal bridges in a life-cycle context are 2.9096 × 10^5 and 4.3037 × 10^5 kg, respectively. The total CO_2 emissions of the UHPC bridges are around 67.5% of that with respect to the normal bridge. Overall, the application of UHPC can aid the sustainable development of infrastructures considering the impacts in a long-time interval.

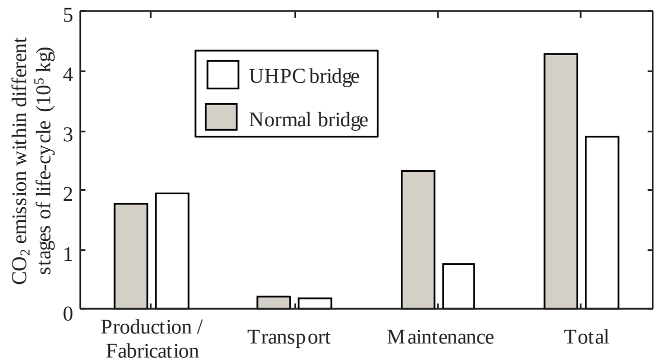


Figure 9. Life-cycle CO_2 emissions of UHPC and normal bridges within 75 years

4.3 Life-cycle cost and equivalent annual cost analyses

The total life-cycle costs of the these two bridges are investigated and compared. In order to compute the life-cycle cost, the investigated time interval and unit material cost should be identified first. The investigted time interval is assumed to be 75 years. The cost of unit UHPC varies and depends on the location of construction site, cost of silica fume, steel fiber, etc. Bascially, the material cost of UHPC is much higher than that of normal concrete due to its very high cement content and steel fiber addition. Herein, the unit cost of UHPC is 830 USD/m^3 and the cost for the nomral concrete is 100 USD/m^3. The cost of UHPC and normal concrete depends on many factors, such as the location of construction site, cost of silica fume, steel fiber, and among others. Herein, the monetary unit cost of UHPC and normal concrete is based on local concrete suppliers. Given more information, other values could also be used in this paper. Based on Eq. (9), the initial construction cost for conventional and UHPC bridges are 1.0607 × 10^5 and 1.9134 × 10^5 USD, respectively. By using the UHPC, the initial construction cost of the bridge will increas approximately 80%. Given the initial cost and the ratio with respect to different maintenance actions, the relevant cost at a given time step is computed. The monetray discount rate used herein is 2%. Then, based on Eq. (7) and the timelines of MR&R mentioned in the previous section, the total MR&R costs of these two bridges are 9.7311×10^5 and 4.0676×10^4 USD, respectively. The life-cycle costs of the nomral and UHPC bridge are 2.0338×10^5 and 2.3201×10^5 USD, respectively. The life-cycle cost of UHPC is larger than that of the conventional bridge. In this paper, the indirect consequence is not incorporated within the evaluation process. Actually, the society and communities would be disturbed by the routines of MR&R activities, leading to adverse social effect. As the use of UHPC can result in a reduction of construction time and number of workers, it has the significant potential to improve construction and repair efficiency. By considering this aspect, the life-cycle cost of the UHPC bridge has the potential to be less than that of the normal bridge.

With respect to the normal bridge and UHPC bridge, the design service life of the UHPC bridge is much larger than that of normal bridge. As these two bridge systems have different service lives, the life-cycle cost as indicated in Eq. (8), can not be used to compare the cost effectiveness of these two bridges. The equivalent annual cost, as a commonly used indicator to compare the cost effectiveness of assets with different service lives, is computed herein. The UHPC bridge is expected to have at least twice the service life of conventional bridge. Accordingly, the investigated the service life for the UHPC bridge is 150 years and the service life for the normal bridge is 75 years. Then, the equivalent annual cost of these two bridges can be computed using Eq. (10), given $r = 2\%$. The equivalent annual cost of these two bridges is 5.2583×10^3 and 5.1899×10^3 USD, respectively. Accoridingly, by considering the equivalent annual cost, the UHPC bridge is more economical and cost benefial. Herein, the residual value and demolish cost are not considered within the life-cycle cost analysis process.

The effect of monetary discount rate on life-cycle cost and equivalent annual cost is conducted herein by varying the monetary discount rate from 0 to 4%. The relevant results are shown in Figure 10(a). As indicated, the life-cycle cost decreases with the increase of the monetary discount rate and difference between the life-cycle costs of these two bridges increases as the monetary discount rate increases. By considering the life-cycle cost, the normal bridge is always chosen as an economic solution. Additionally, the effects of the investigated time interval on the life-cycle cost are shown in Figure 10(b). As indicated, the normal bridge remains as the cost-effective option when the investigated time interval is below 120 years. Given the investigated time interval larger than around 120 years, the life-cycle cost of the normal bridge is larger than the that of the UHPC bridge.

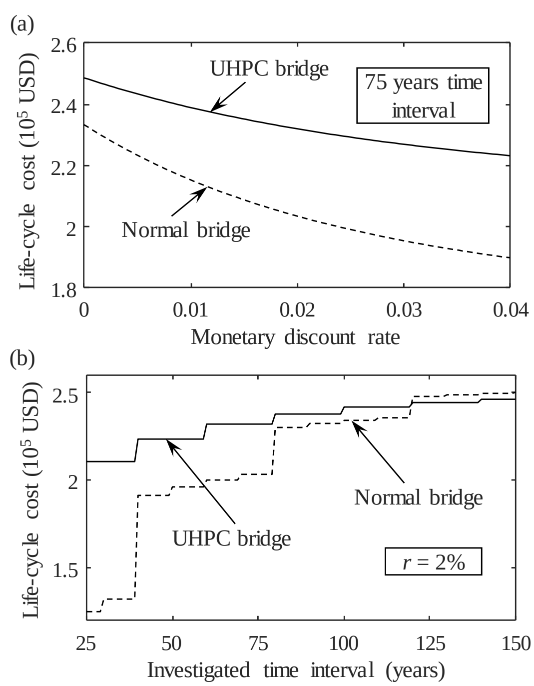


Figure 10. Effect of (a) monetary discount rate and (b) investigated time interval on the life-cycle cost of normal and UHPC bridges

Given the service life of the normal bridge is 50 years, the life-cycle cost and equivalent annual cost of UHPC bridge under different service lives are shown in Figure 11. As indicated, the life-cycle cost of UHPC bridge is always higher than that of the normal bridge. The normal bridge would always be the chosen option by considering the life-cycle cost under this circumstance, while different conclusion is obtained if the equivalent annual cost is emphasized. If the service life of the UHPC bridge is larger than approximate 62 years, the UHPC bridge would be a more economical option. Additionally, the equivalent annual cost of the UHPC bridge given the service life of the normal bridge is 75 years is shown in Figure 12. Considering the high corrosion-resistant and durability of UHPC, the UHPC structure could have a more significant advantage over the structures using the normal concrete, especially for the structures located in an aggressive environment. Given more detailed information, the benefit of UHPC bridges located in different environmental scenarios can be computed.

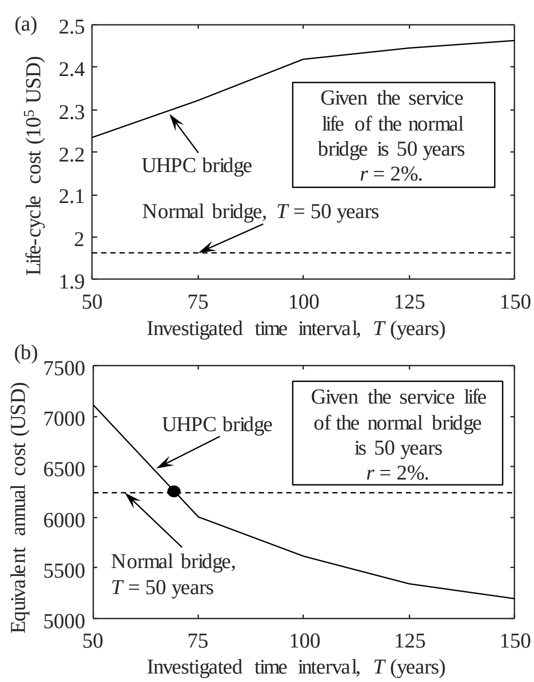


Figure 11. (a) Life-cycle cost and equivalent annual cost of UHPC bridge under different time intervals given the service life of the normal bridge is 50 years and $r = 2\%$

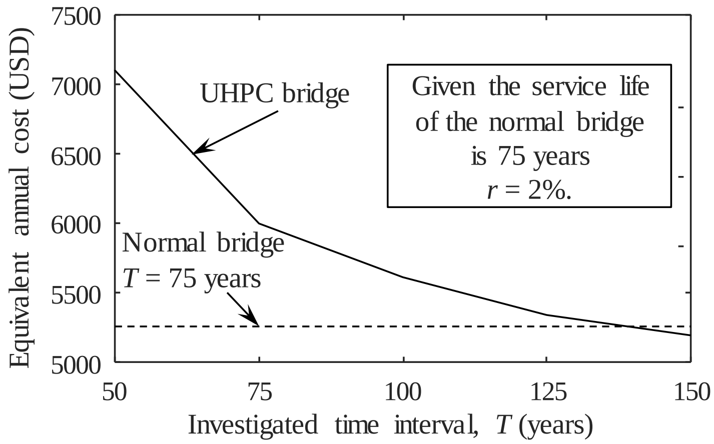


Figure 12. Equivalent annual cost of UHPC bridge under different time intervals given the service life of the normal bridge is 75 years and $r = 2\%$

5. Conclusions

This paper provides a systematic approach for the life-cycle assessment of conventional and UHPC bridges considering environmental impacts, cost, and equivalent annual cost. It is illustrated by considering normal and UHPC bridges with same reliability index. The approach presented in this paper is not only aimed to assess the performance of UHPC structures in a life-cycle context, but it is also aimed to facilitate the further application of UHPC within the civil infrastructures.

This study has demonstrated that the structures using UHPC are associated with significant benefit in terms of sustainable perspective. The benefit of using UHPC is not obvious during the construction phase, while the reduction of environmental impacts in terms of CO_2 emissions is reduced approximately 48% due to the less MR&R actions are needed compared with the normal concrete structures. Accordingly, using of UHPC can be seen as one of the key options to mitigate the CO_2 emissions and combat the climate change. In this study, it has been shown that the use of UHPC has lower global warming impact compared with normal concrete structures when service life is considered.

By using the UHPC, the initial construction cost of the bridge increases approximately 80%. The life-cycle cost of these two bridges are also investigated. Accordingly, the life-cycle cost of UHPC is larger than that of the conventional bridge when the service life of these two bridges is less than approximate 120 years. By comparing the bridges with different service lives using UHPC and normal concrete, the equivalent annual cost is used. By considering the equivalent annual cost, the cost of the UHPC bridge can be smaller than that of the normal bridge, as the UHPC bridge can extend the structural service life. By considering the life-cycle cost and equivalent annual cost, different decisions associated with the UHPC and normal bridges can be made.

Further studies are needed to investigate both the short- and long-term performance of UHPC, as it is different from the normal concrete and a large uncertainty can be involved. For instance, the post-cracking tensile strength of UHPC depends on many parameters (e.g., distribution and orientation of embedded fibers) and can result in a large scattering response, while these aspects are not well considered by current specification. Thus, more studies are needed to assess the performance of UHPC structures by considering different parameters (e.g., uncertainties within mechanical properties, design service life, long-term performance) to update the current design specification and to aid the reliability-informed design, assessment, and management of UHPC structures. Additionally, the monetary value of CO_2 emissions could also be investigated and incorporated within the life-cycle cost analysis.

Use of UHPC can provide a green and sustainability solution for the development of the next generation of bridges to address the sustainable development issues. Based on this study, UHPC has been approved to efficiently prolong the service life, improve the environmental impact in a long term, and reduce the equivalent annual cost. In addition, performance of UHPC bridges should be compared with other structural systems (e.g., steel bridge) to aid widely practical application of UHPC within civil engineering. This study can bridge the gap existed between engineers developing new materials and designers who will ultimately use them in practice and aims to develop novel elements and structure to achieve the sustainability goals in a life-cycle context by using high performance materials.

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