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Investigation of the Benefit of Using Novel Corrosion-resistant Steel in New and Existing Bridges in Pennsylvania

March 10, 2022

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Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
CIAM-UTC-REG6		

4. Title and Subtitle	5. Report Date
Investigation of the Benefit of Using Novel Corrosion-resistant Steel in New and Existing Bridges in Pennsylvania	March 10, 2022
	6. Performing Organization Code
7. Author(s) Dan M. Frangopol, https://orcid.org/0000-0002-9213-0683 Xu Han, https://orcid.org/0000-0003-2136-5466	**8. Performing Organization Report No.**
9. Performing Organization Name and Address ATLSS Engineering Research Center Department of Civil and Environmental Engineering, Lehigh University 117 ATLSS Drive Bethlehem, PA 18015	**10. Work Unit No. (TRAIS)** **11. Contract or Grant No.** 69A3551847103
12. Sponsoring Agency Name and Address U.S. Department of Transportation Research and Innovative Technology Administration 3rd Fl, East Bldg E33-461 1200 New Jersey Ave, SE Washington, DC 20590	**13. Type of Report and Period Covered** Final Report 02/10/2021 – 03/10/2022 **14. Sponsoring Agency Code**

15. Supplementary Notes

Work funded through The Pennsylvania State University through the University Transportation Center Grant Agreement, Grant No. 69A3551847103.

16. Abstract

For a steel bridge subjected to deterioration mechanisms like corrosion, different types of maintenance actions need to be considered to fully/partially restore the functionality of the bridge in an effective manner. In the decision-making process of risk-based optimal life-cycle maintenance strategy, the optimal maintenance solutions are influenced by multiple factors, such as the type of maintenance actions applied and correlation among girder resistances. In addition, operational dependency is a factor that needs to be accounted for in the decision-making process of optimal maintenance strategies. Some maintenance actions are hard to implement in real engineering projects, and therefore should not be considered as feasible maintenance actions. Parametric analysis is worth carrying out to shed light on the influence of these parameters on optimal maintenance plans. For cases where girder replacement using corrosion-resistant steel is adopted as the maintenance action, the maintenance effect of optimal maintenance solutions can show the cost-effectiveness of corrosion-resistant steel. In the risk-based optimal decision-making process for multi-girder steel bridges subjected to corrosion, when and which part of the superstructure needs to be replaced during one maintenance action should be determined through optimization. The results of the optimization indicate that using A709-50CR can achieve economical benefits when operational dependency is considered.The cost-effectiveness of using corrosion-resistant steel such as A709-50CR to replace corroded carbon steel girders should not only be investigated at an individual bridge level, but also at a bridge network level. Investigations on the optimal maintenance strategy for infrastructure systems such as bridge networks have been gaining momentum as the importance of the concept of infrastructure systems gains recognition. Using an existing bridge network in Pennsylvania, it is shown that the economic benefit of risk-based life-cycle maintenance at a bridge network level may outweight the sum of benefit of the maintenance actions applied on each individual bridge in the network.

17. Key Words	18. Distribution Statement
Corrosion, risk, optimization, A709-50CR, bridge network	No restrictions. This document is available from the National Technical Information Service, Springfield, VA 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	39	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

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CHAPTER 1

Introduction

BACKGROUND

In Pennsylvania, the average age of bridges recorded in the National Bridge Inventory is 40 years. Among these bridges, simply supported non-composite steel bridges are considerably older than many other bridge types. According to the bridge data provided by the Pennsylvania Department of Transportation (PennDOT), the average age of these bridges on the State Route System has reached 53 years, while over 50% of them are older than 63 years. Considering the fact that most bridges designed in the period 1950s-1970s have a design service life of 50 years, most steel girder bridges in PA have been servicing beyond their initial design limits. These bridges serve an overall average daily traffic (ADT) of around 2.5 million vehicle units. Obviously, this poses considerable risk to the functionality of the transportation infrastucture and to the safety of millions of traffic users. Nevertheless, a total overhaul and rebuilding of all these bridges is neither realistic from a budgetary standpoint nor reasonable given that over-design is prevalent in bridges. This dilemma regarding aging bridges is not unique to Pennsylvania but is also common in other Mid-Atlantic (Region 3) states. Therefore, the use of novel materials such as corrosion-resistant steel in new and existing bridges is urgently needed to ensure that the risk of failure of steel girder bridges will be under an accetable level.

The objective of this project was to investigate the benefit of using a novel corrosion-resistant steel, namely A709-50CR, for steel girder bridges in Pennsylvania. Cost-effectiveness of using A709-50CR girders to replace corroded carbon steel girders for steel bridges subjected to corrosion has been investigated in Frangopol *et al.* (2020). The results show that using A709-50CR girders to conduct replacement can lead to a reduced maintenance budget compared with using new carbon steel girders under stringent requirement of life-cycle risk. As optimal maintenance solutions for steel bridges subjected to corrosion are influenced by various parameters, a comprehensive parametric analysis of these influence factors needs to be conducted in order to have a sound understanding of the cost-effectiveness of using A709-50CR for girder replacement. In addition, operational dependency can be considered as an influence factor of the optimal maintenance solutions. As reseach on the life-cycle analysis of infrastructure systems is gaining momentum, the research scope of the application of corrosion-resistant steel in bridge maintenance actions should be extended from the individual bridge project level to the bridge network level.

OBJECTIVES

There were two main goals of this research. One was to further investigate the cost-effectiveness of using A709-50CR girders to conduct girder replacement considering different influence factors. The other was to investigate the cost-effectiveness of using A709-50CR girders on a bridge network level. Specifically, the objectives of this research were:

- To investigate the influence of cost premium of A709-50CR over carbon steel (i.e., the percentage by which A709-50CR's cost exceeds that of carbon steel) on the cost-effectiveness of A709-50CR compared with carbon steel,

- To investigate the influence of correlation among girder resistances in an individual bridge on the cost-effectiveness of A709-50CR compared with carbon steel,
- To investigate the influence of system models of one bridge on the cost-effectiveness of A709-50CR compared with carbon steel,
- To investigate the influence of selected performance indicators on the cost-effectiveness of A709-50CR compared with carbon steel, and
- To investigate the cost-effectiveness of A709-50CR over carbon steel at a bridge network level.

DATA AND DATA STRUCTURES

Data used in this report include corrosion rate data of carbon steel and data on A709-50CR steel. The data were summarized in Han *et al.* (2021b). Traffic volume data from PennDOT (2021) are utilized herein to conduct traffic analysis. The authors would like to thank Mr. Thomas E. Macioce, P.E., from the Pennsylvania Department of Transportation for providing the bridge drawings used in the study of the individual bridge in Montgomery County, PA, and the bridge network in Chester County, PA.

CHAPTER 2

Methodology

ABSTRACT

Optimal maintenance solutions associated with girder replacement using either A709-50CR or carbon steel girders are obtained herein through optimization processes. Corrosion modeling is considered to obtain time-variant girder resistances as well as time-variant correlation coefficients among girder resistances. Performance functions of girders are established based on AASHTO specifications (AASHTO 2017). System reliability analysis is then conducted to obtain time-variant system reliability profiles. Failure consequence evaluation is conducted to obtain time-variant risk profiles. Genetic algorithm is utilized to determine the optimal maintenance solutions for individual bridges or bridge networks.

PARAMETRIC ANALYSIS ON INFLUENCE FACTORS OF THE COST-EFFECTIVENESS OF A709-50CR

Influence factors of cost-effectiveness of A709-50CR

As mentioned previously, cost-effectiveness of A709-50CR is contingent upon many factors. The most direct factor is the cost premium of this novel corrosion-resistant steel over carbon steel. It is self-evident that the cost-effectiveness of A709-50CR will decrease if the cost premium of A709-50CR over carbon steel increases. Time-variant correlation among girder resistances can influence time-variant risk profiles for bridges, and therefore can influence the cost-effectiveness of A709-50CR. System models are associated with the redundancy of bridge structures, which characterizes the relationship between the failure of individual girders and failure of the entire bridge. Under the same reliability profile for individual girders, different system models can result in different system reliability profiles for the entire bridge. Therefore, system models can influence time-variant risk profiles and hence influence the cost-effectiveness of A709-50CR. Last but not least, the performance indicator chosen by the decision-makers may make a difference in the optimal solutions under a specific maintenance budget. Reliability and risk are two widely adopted performance indicators in life-cycle analysis. Which of these two performance indicators is chosen by decision-makers may influence the cost-effectiveness of A709-50CR.

Information about the steel bridge investigated

The bridge investigated herein is a multi-girder steel bridge located in Montgomery County, PA. The bridge is a composite bridge consisting of four steel girders topped by the concrete deck. Information about this bridge to obtain time-variant risk profiles has been given in Frangopol *et al.* (2020), including geometric configuration of the bridge, performance functions associated with flexural and shear failure modes of the girders, and random variables associated with corrosion modeling, resistance calculation, and load effects, among others.

Correlation

For the steel bridge investigated, it is assumed that within one girder, random variables associated with a specific material/corrosion property at different regions are fully correlated, such as the yield strength of steel at different cross sections and the corrosion rate at different regions in one girder, among others. Random variables associated with different material/corrosion properties are independent. Independence also holds for random variables associated with material properties and random variables associated with corrosion properties. For random variables of different girders associated with the same material/corrosion property, it has been determined in Frangopol *et al.* (2020) that random variables associated with four major properties play a dominant rule in characterizing the correlation coefficient among inter-girder resistances (i.e., resistances of different girders), namely the yield strength of steel f_y, strength of concrete f_c, coating life t_0 in the same painting/repainting action, and corrosion rate r_{corr} of different girders. In Frangopol *et al.* (2020), a common correlation coefficient ρ_X was adopted to denote the correlation among random variables associated with each of these four properties. For a four-girder steel bridge investigated herein, the correlation matrix of random variables associated with each of four properties is

$$\boldsymbol{\rho_X} = \begin{pmatrix} 1 & \rho_X & \rho_X & \rho_X \\ \rho_X & 1 & \rho_X & \rho_X \\ \rho_X & \rho_X & 1 & \rho_X \\ \rho_X & \rho_X & \rho_X & 1 \end{pmatrix} \quad (1)$$

where $\boldsymbol{\rho}_X$ is the correlation matrix and X can be f_c, f_y, t_0, or r_{corr}.

ρ_X is assumed to be 0.9 in Frangopol *et al.* (2020), which indicates a high correlation among girder resistances. In this report, parametric analysis is conducted on ρ_X to investigate the influence of correlation among girder resistances on the time-variant risk profiles and optimal maintenance solutions associated with girder replacement. Three different correlation cases are considered:

- Case I: $\rho_X = 0.9$ for all four random variables,
- Case II: $\rho_X = 0.1$ for all four random variables, and
- Case III: $\rho_X = 0.1$ for $\rho_X = 0.9$ for $X = f_c$ and f_y; $\rho_X = 0.1$ for $X = t_0$ and r_{corr}

Case I is the same as the correlation scenario considered in Frangopol *et al.* (2020). Case II is the same as Case I except that the level of correlation is low. In Case III, high correlation is assumed for f_c and f_y in different girders, while low correlation is assumed for t_0 and r_{corr} in different girders. For the investigation on time-variant correlation coefficients, the bridge failure is defined as the failure of any two adjacent girders.

System modeling

Multi-girder bridges may fail when an individual girder fails due to excessive load effects, or when several adjacent girders reach their ultimate load-carrying capacity due to load-sharing effects. A multi-girder bridge can be represented as a system of girders, in which several adjacent girders can be considered as sub-parallel systems, and different groups of adjacent girders are in series with each other (Estes 1997). The exact number of failed girders that can lead to the failure of the entire bridge is dependent upon factors such as the stiffness of the bridge deck and the residual strength of failed girders (Rosowsky & Ellingwood 1991; Enright & Frangopol 1999). It is hard to generalize a system model for girder bridges, as uncertainties of material properties lead to uncertainties of load redistribution after one girder fails. Given inadequate knowledge on the exact number of adjacent girders, the failure of which can result in the failure of the entire bridge, a sensitivity analysis is performed herein on this number (i.e., the number of adjacent girders involved in a sub-parallel system). Three models are considered herein: (a) the bridge superstructure is a series model (Model I), (b) any two adjacent girders' failure results in the bridge failure (Model II), and (c) any three adjacent girders' failure results in the bridge failure (Model III).

Performance indicators

Two different life-cycle maintenance strategies with different performance indicators have been applied extensively, namely the reliability-based life-cycle management and risk-based life-cycle management. The concept of reliability-based life-cycle management was proposed in the late 1990s (Thoft-Christensen 1999; Frangopol *et al.* 2001). Reliability as a performance indicator has gained wide application, as it is a much more accurate indicator to quantify bridge performance than discrete bridge conditions (the state-of-the-art performance indicator before the concept of reliability was proposed), and reliability is a more appropriate indicator in assessing the bridge performance at a system level (instead of individual bridge components). Risk-based life-cycle management incorporates both reliability analysis and failure consequence estimation processes. Risk, as a performance indicator, is defined as the product of probability of failure and failure consequences (Ang & Tang 1984). Risk assessment and risk-based design are gaining popularity as it takes into account the fact that some structural failure events have more severe consequences than others. Risk-based analysis lends itself to structures with a small failure probability and large failure consequences. In addition, risk assessment is an essential step in performance-based design (PBD), which is a trending design philosophy to tackle disasters such as seismic or fire hazard (Ghosn *et al.* 2016; Lounis & McAllister 2016).

A multi-objective optimization process can be involved in both reliability- and risk-based life-cycle management frameworks. As maximizing the bridge performance and minimizing the cost of intervention actions are in conflict with each other, the Pareto optimality (Pareto 2014) has often been adopted, where a solution is called a Pareto optimum if no other feasible solutions can result in an improvement of some criteria without worsening at least one of the other criteria. In this study, the reliability- and risk-based life-cycle management strategies are considered in the maintenance of carbon steel bridges subjected to corrosion. Girder replacement using A709-50CR/carbon steel is considered an essential maintenance action and girder repainting is considered a preventive maintenance action. Following the approach in Frangopol *et al.* (2020), a regular time interval of 25 years is scheduled for the repainting actions. Girder replacement can interrupt the prescribed repainting action. When and which part of steel girders in the bridge superstructure to be replaced are design variables in the optimization process. The optimization is bi-objective. The first objective is to minimize the total life-cycle cost, which includes the initial building cost and life-cycle maintenance cost. The second objective is to minimize life-cycle failure probability in the case of reliability-based life-cycle maintenance strategy and to minimize life-cycle risk in the case of risk-based life-cycle maintenance strategy.

The expression for the total life-cycle cost is

$$C_{tot} = C_{ini} + \sum_{i=1}^{N_{mx}} \frac{C_{rep,i}}{(1+r)^{t_i}} \tag{2}$$

where C_{ini} is the initial construction cost; N_{mx} is the number of maintenance actions; $C_{rep,i}$ $(i = 1 \dots N_{mx})$ is the repair cost of the ith maintenance action; t_i $(i = 1 \dots N_{mx})$ is the time of the ith maintenance action.

The expression for the life-cycle failure probability P_{flc} is

$$P_{flc} = 1 - \prod_{t=1}^{T} (1 - P_f(t)) \tag{3}$$

where $P_f(t)$ is the annual failure probability at year t; T is the total life span of the structure.

The life-cycle risk of a bridge can be expressed as

$$R_f = \sum_{t=1}^{T} P_f(t) \cdot \frac{C_A}{(1+rr)^t} \tag{4}$$

where C_A is the failure consequence; rr is the discount rate of money.

Operational dependency

Given the consideration of operational dependency, based on the rational replacement scheme proposed in previous studies (Estes & Frangopol 1999), four replacement schemes are considered as rational for a multi-girder steel bridge superstructure: (a) replacing two exterior girders, (b) replacing all the interior girders, (c) replacing girders in the left/right half of the bridge superstructure in the transverse direction, and (d) replacing all girders of the bridge superstructure. Another criterion for checking if the replacement schedule is rational is that girder replacement should not be conducted if the girder was recently painted. In addition, for life-cycle analysis within the design service life, major maintenance actions like girder replacement at the beginning or near the end of bridge service life are considered unrealistic. In this case, girder replacement actions for the bridge when its service life is smaller than 10 years or larger than 65 years (the design service life of the bridge is 75 years) are deemed as infeasible. The minimum time interval between one girder replacement action and the previous repainting action on the same girder is set at 15 years. Another constraint added is that the time interval between two consecutive replacement actions should not be smaller than 10 years. Last but not least, a minimum annual reliability threshold of 3.0 is set for flexural and shear failure modes of individual girders (AASHTO 2018), as it is not reasonable to replace a carbon steel girder when its resistance has dropped to a minimal level due to corrosion.

Two options are considered for the replacement of corroded carbon steel girders: one is to conduct girder replacement using carbon steel girders, the other is to use A709-50CR girders for replacement. The repainting cycles of 25 years are reset from the time of replacement when carbon steel girders are used. For girder replacement using A709-50CR girders, as the corrosion rate of this new type of steel is extremely small, no further repainting is needed.

The constraints are formalized as follows :

- The interval of two consecutive girder replacement actions $\Delta t \geq 10.0$ years,
- The time of the first maintenance $t_1 \geq 10$ years,
- The time of the last maintenance $t_{end} \leq T_b - 10$ years,
- $\boldsymbol{\beta}_{min,annual} \geq \boldsymbol{\beta}_{\mathbf{thr}}$, where $\boldsymbol{\beta}_{min,annual} = \{\beta_{min,1}, \beta_{min,2}, \dots \beta_{min,N}\}$ is the vector containing the minimum component annual reliability indices among the N failure modes of all girders considered; $\boldsymbol{\beta}_{thr} = \{\beta_{thr,1}, \beta_{thr,2}, \dots \beta_{thr,N}\}$ is the vector of target reliability indices associated with component ultimate limit states,
- The time between essential maintenance action and the previous preventative maintenance action $t_{ess,i} - t_{prev,i} \geq 15$ years, $i = 1 \dots N_{mx,ess}$,
- The girders that are replaced during one essential maintenance must conform to previously determined rational operational strategy.

T_b refers to the life span of the bridge; $N_{mx,ess}$ is the number of essential maintenance actions. Nondominated Sorting Genetic Algorithm II (NSGA-II) (Deb *et al.* 2002) is adopted to perform the optimization using MATLAB global optimization toolbox (MathWorks 2018).

In addition to the optimal maintenance solutions obtained through the genetic algorithm, cost-effectiveness of another two maintenance strategies is considered herein. One is to repaint the carbon steel girders at a time interval of 15 years and not to conduct any girder replacement. The 15-year repainting interval is determined based on the coating life inferred from the NBI database (FHWA 2018). The coating life inferred from NBI database (FHWA 2018) is modeled by a lognormal distribution with a mean of 15.3 years and a standard deviation of 5.3 years. The other is to build an A709-50CR bridge, which means that the A709-50CR is used as the material for steel girders during initial construction.

Maintenance cost and failure consequences

Among the maintenance actions considered above, replacing the entire superstructure using carbon steel is estimated to cost 8.02×10^5 USD (Estes & Frangopol 1999), while the cost of replacing two girders (corresponding to replacement options a, b, and c) is assumed to be 80% of the superstructure replacement cost (corresponding to replacement option (d). Repainting of a carbon steel girder costs around 15% of the girder cost (FHWA 2011), which equals to 3.01×10^4 USD based on the overall superstructure cost. The initial superstructure construction cost for the bridge is estimated to be 75% of the superstructure replacement cost (Estes & Frangopol 1999; Ford *et al.* 2012). As mentioned previously, cost premium of A709-50CR over carbon steel is also an influence factor on the cost-effectiveness of A709-50CR. As A709-50CR is a relatively new construction material, its cost is subjected to fluctuations, and estimation of its cost premium over carbon steel has a considerable dispersion. Cost volatility of A709-50CR is considered herein using three representative cost premiums obtained from the existing literature. They are low-cost premium at 7.9% (Soliman & Frangopol 2015), medium-cost premium at 16.4% (Kogler 2015), and high-cost premium at 57.9% (Hebdon 2018). Annual discount rate of money is assumed to be 2%. The failure consequence associated with bridge collapse is estimated at 6.90×10^7 USD, including 1.09×10^6 USD rebuilding cost, 1.95×10^7 USD cost to traffic users, and 4.84×10^7 USD fatality cost (Han *et al.* 2021a).

COST-EFFECTIVENESS OF A709-50CR ON A BRIDGE NETWORK LEVEL

Concept of bridge network

A transportation network that consists of multiple nodes (traffic junctions) and links (traffic lines) is an essential type of infrastructure system. Bridges are considered as essential components in the network, as the failure of a bridge will render the associated link out of function, causing huge economic losses due to the traffic disruption. The concept of bridge network performance (Liu & Frangopol 2006), defined as the performance of a highway transportation network characterized by nodes as well as other elements in the transportation network such as pavements, is adopted herein. Among all these elements in the bridge network, the safety of the bridges is prioritized over the safety of other elements. As bridges are prone to corrosion attack, the functionality of the entire bridge network decreases with time if no maintenance is applied.

Different bridges in the bridge network are intertwined components, as the failure of one bridge may affect the traffic volume on another bridge. User cost associated with extra travel time and extra travel distance when bridge failures occur is an essential aspect of failure consequence. Traffic analysis needs to be carried out to determine how the traffic volume changes on each link in order to estimate the user cost on a bridge network level. For a specific path in a bridge network, the nodes at the beginning and the end can form an O-D (origin-destination) pair. An O-D matrix can provide information on the number of travelers associated with each O-D pair. Elements in the O-D matrix can be estimated based on the average daily traffic data of the links in the bridge network (Van Zuylen & Willumsen 1980).

Travelers' path choice behavior, i.e., which path travelers will take when multiple paths are available for an O-D pair, can also influence the traffic volume on each link when the O-D matrix is fixed. Different user equilibrium approaches have been proposed to characterize travelers' behavior (Bell & Iida 1997). These approaches can be categorized into two major types, namely the deterministic user equilibrium approach and stochastic user equilibrium approach. The deterministic user equilibrium approach assumes that traffic users will definitely select the least-cost path (the path with the minimum travel time) when facing multiple path choices. The stochastic user equilibrium approach acknowledges the fact that due to the imperfect knowledge of traffic users on the path cost, the least-cost path will only be selected by the traffic users with a higher probability than higher-cost paths. It should be acknowledged that the stochastic user equilibrium approach solves the traffic volume distribution within a bridge network in a more realistic manner than the deterministic user equilibrium approach. Among all the traffic assignment models associated with the stochastic user equilibrium approach, the logit assignment model (Dial 1971) is a widely used model that can be integrated into the stochastic user equilibrium calculation. For a specific O-D pair with n paths, the probability that a specific path k is selected by traffic users based on the logit assignment is (Bell & Iida 1997)

$$P_k = \frac{\exp(-\theta C_k)}{\sum_{j=1}^{n} \exp(-\theta C_j)} \tag{5}$$

where C_k is the travel cost on path k; C_j is the travel cost on path j; n is the number of available paths; θ is the dispersion factor characterizing traffic users' sensitivity to the travel cost. A very large θ (e.g., larger than 10^3) indicates that traffic users will choose the least cost path, while $\theta = 0$ indicates that traffic users will choose all the paths with an equal probability.

Risk-based optimal life-cycle maintenance for bridge networks

Similar to the life-cycle maintenance actions for individual bridges, timing of maintenance actions of each bridge in a bridge network constitutes an optimization problem (Bucher & Frangopol 2006; Okasha & Frangopol 2010; Frangopol & Bocchini 2012). Optimization is carried out herein to determine when to replace the superstructure of the deteriorating bridges in a bridge network. The risk-based optimization for a bridge network is bi-objective. The first objective is to minimize the total life-cycle maintenance cost for the bridge network investigated. The second objective is to minimize the life-cycle risk of the network.

The same types of maintenance actions considered for the steel bridge in Montgomery County, PA, were considered for the steel bridges in the bridge network by applying a repainting action every 25 years as preventive maintenance actions and girder replacement may interrupt the repainting schedule as essential maintenance actions. The girder replacement can be conducted using either carbon steel girders or A709-50CR girders. For the concrete bridges in the bridge network, girder replacement using new girders with the same geometry and mechanical properties as that of old girders is considered. The expression for the total life-cycle maintenance cost C_{mx} is

$$C_{mx} = \sum_{ii=1}^{N_{rep}} \sum_{jj=1}^{N_b} \frac{C_{rep,ii,jj}}{(1+rr)^{t_{ii}}} + \sum_{kk=1}^{N_{repaint}} \sum_{jj=1}^{N_b} \frac{C_{repaint,kk,jj}}{(1+rr)^{t_{kk}}} \tag{6}$$

where N_{rep} and $N_{repaint}$ are the number of replacement and repainting actions within the life-cycle of the bridge network, respectively; N_b is the number of bridges in the network; t_{ii} ($ii = 1, 2, \ldots, N_{rep}$) and t_{kk} ($kk = 1, 2, \ldots, N_{repaint}$) are the times of the iith replacement action and the kkth repainting action, respectively; $C_{rep,ii,jj}$ and $C_{repaint,kk,jj}$ ($ii = 1, 2, \ldots, N_{rep},\ jj = 1, 2, \ldots, N_b, kk = 1, 2, \ldots, N_{repaint}$) are the replacement costs of the superstructure of the jjth bridge in the iith replacement action and repainting cost of the superstructure of the jjth bridge in the kkth repainting action, respectively; $C_{rep,ii,jj}$ is zero if the superstructure of the jjth bridge is not replaced in the iith replacement action and $C_{repaint,kk,jj}$ is zero if steel girders of the jjth bridge are not involved in the kkth repainting action.

Failure consequences need to be determined for the bridge network in order to obtain life-cycle risk at the network level. Both direct and indirect failure consequences are considered herein. Direct failure consequence refers to the cost associated with bridge reconstruction, which is usually estimated on a per-unit deck area basis. Indirect cost herein refers to the user cost associated with extra travel time and extra travel distance, which entails network analysis. The rebuilding cost of a certain bridge is expressed as

$$C_{bridge,jj} = c_{bridge,jj} W_{jj} L_{jj} \tag{7}$$

where $C_{bridge,jj}$ is the total rebuilding cost of the jjth bridge ($jj = 1,2, \ldots N_b$); $c_{bridge,jj}$ is the rebuilding cost per unit deck area for the jjth bridge; and W_{jj} and L_{jj} are the width and length of the jjth bridge, respectively.

As the unit user cost per truck is different from that per car, the user costs associated with these two types of vehicles are calculated separately and then combined in the user cost estimation process. The user cost due to extra total travel time and extra travel distance is expressed as

$$C_{time,jj} = c_{AW}O_{car} \cdot \left(TTT_{jj,car} - TTT_{0,car}\right) + \left(c_{ATC}O_{truck} + c_{good}\right) \cdot (TTT_{jj,truck} - TTT_{0,truck}) \tag{8a}$$

$$C_{run,jj} = c_{run,car} \cdot \left(TTD_{jj,car} - TTD_{0,car}\right) + c_{run,truck} \cdot (TTD_{jj,truck} - TTD_{0,truck}) \tag{8b}$$

where $C_{time,jj}$ and $C_{run,jj}$ are the user cost due to extra travel time and travel distance associated with the failure of the jjth bridge ($jj = 1,2, \dots N_b$) in the network, respectively; $TTT_{jj,car}$ and $TTT_{jj,truck}$ are the total travel time of cars and trucks conditioned upon the failure of the jjth bridge ($jj = 1,2, \dots N_b$) in the network, respectively; $TTT_{0,car}$ and $TTT_{0,truck}$ are the total travel time of cars and trucks in the original network (where all the links are in an intact state), respectively; c_{AW} and c_{ATC} are the average compensation for car drivers and truck drivers per unit time period, respectively; c_{good} is the time value of goods on the trucks; O_{car} and O_{truck} are average occupancy for cars and trucks, respectively; $TTD_{jj,car}$ and $TTD_{jj,truck}$ are the total travel distance of cars and trucks conditioned upon the failure of the jjth bridge ($jj = 1,2, \dots N_b$) in the network, respectively; $TTD_{0,car}$ and $TTD_{0,truck}$ are the total travel distance of cars and trucks in the original network (where all the links are in an intact state), respectively; $c_{run,car}$ and $c_{run,truck}$ are the average running cost of cars and trucks, respectively. Based on Decò and Frangopol (2011), the average compensation for car drivers and truck drivers are 22.82 USD/h and 22.97 USD/h, respectively. The average vehicle occupancies for cars and trucks are 1.5 and 1.05, respectively. The running costs of cars and trucks are 0.08 USD/km and 0.375 USD/km, respectively. Time value of a cargo loaded in trucks is 4 USD/h.

The total travel time (TTT) and total travel distance (TTD) are expressed as

$$TTT = \sum_{a \epsilon \mathcal{A}} f_a t_a(f_a) \tag{9a}$$

$$TTD = \sum_{a \epsilon \mathcal{A}} f_a s_a \tag{9b}$$

where f_a is the traffic flow on link a; s_a is the length of link a; t_a is the travel time on link a (expressed as a function of f_a); $\mathcal{A}$ is the set of operational links.

Expression of t_a given in Bureau of Public Roads (1964) is

$$t_a = t_{a,0}\left[1 + \alpha\left(\frac{f_a}{f_{a,c}}\right)^{\beta}\right] \tag{10}$$

where α and β are parameters the value of which is equal to 0.15 and 4, respectively (Bocchini & Frangopol 2011); $t_{a,0}$ is the free speed travel time on link a; $f_{a,c}$ is the traffic capacity of link a.

The downtime (or the reconstruction time) of bridges needs to be determined in the user cost calculation. Through regression analysis in Jiang and Wu (2004), the average reconstruction time T_{down} can be expressed as a function of reconstruction cost C_{bridge} (Jiang & Wu 2004)

$$T_{down} = \alpha_1 \cdot \ln\left(C_{bridge}\right) - \gamma_1 \tag{11}$$

where α_1 and γ_1 are coefficients of regression analysis. For the bridges in state roads, the values of α_1 and γ_1 are taken as 30.86 and 310, respectively (Jiang & Wu 2004).

It is assumed herein that high correlation exists for the failure events of different girders in the same bridge while the correlation among the failure events of different bridges is weak. The annual risk of the bridge network can be approximated as

$$R_{ann}(t) = \sum_{jj=1}^{N_b} \hat{P}_{jj,ann}(t) \cdot (C_{bridge,jj} + C_{time,jj} + C_{run,jj}) \tag{12}$$

where $R_{ann}(t)$ is the annual failure risk at year t; $C_{bridge,jj}$ is the rebuilding cost of the jjth bridge ($jj = 1,2, \ldots N_b$); $C_{time,jj}$ and $C_{run,jj}$ are the user costs due to extra travel time and travel distance associated with failure of the jjth bridge, respectively. $\hat{P}_{jj,ann}(t)$ is the annual unconditional failure probability of the jjth bridge at year t, which can be calculated as (Yang *et al.* 2021)

$$\hat{P}_{jj,ann}(t) = P_{jj,ann}(t) \cdot \prod_{\tau}^{t-1} (1 - P_{jj,ann}(\tau)) \tag{13}$$

where $P_{jj,ann}(\tau)$ is the conditional annual failure probability at year τ (given that the jjth bridge did not fail before year τ), which is obtained using system reliability analysis.

The life-cycle network risk is calculated as

$$R_{lfc} = \sum_{t=1}^{T_{net}} \frac{R_{ann}(t)}{(1 + rr)^{t-1}} \tag{14}$$

where T_{net} is the lifespan of the network.

A constraint is added herein that no bridge replacement action is carried out for an individual bridge in the bridge network within 10 years from the end of its service life. The bi-objective optimization is formulated as:

Given

- Number of bridge superstructures considered for replacement N_b
- Annual reliability profiles of bridges in the network $\boldsymbol{\beta}(t) = \{\beta_1(t), \beta_2(t), \cdots, \beta_{N_b}(t)\}$
- Rebuilding cost of bridges in the network $\boldsymbol{C_{bridge}} = \{C_{bridge,1}, C_{bridge,2}, \cdots, C_{bridge,N_b}\}$
- User cost associated with failure of bridges in the network $\boldsymbol{C_{user}} = \{C_{time,1}, C_{time,2}, \cdots, C_{time,N_b}, C_{run,1}, C_{run,2}, \cdots, C_{run,N_b}\}$
- Maintenance cost $\mathbf{C} = \{C_{rep,1}, C_{rep,2}, \cdots, C_{rep,N_b}, C_{repaint,1}, C_{repaint,2}, \cdots, C_{repaint,N_b}\}$, where $C_{rep,jj}$ and $C_{repaint,jj}$ ($jj = 1, 2, \ldots N_b$) is the replacement cost and repainting cost of the jjth bridge, respectively; $C_{repaint,jj}$ is zero if the jjth bridge is a prestressed concrete bridge
- Existing service life (i.e., time in service at the current year) $\mathbf{T_{ini}} = \{t_{ini,1}, t_{ini,2}, t_{ini,3}, \cdots, t_{ini,N_b}\}$, where $t_{ini,jj}$ is the existing service life for the jjth bridge ($jj = 1, 2, \ldots N_b$)
- Target service life of the network T_{net} (i.e., time in service after the current year)
- Monetary discount rate rr

Find

- Maintenance times of all girders $\mathbf{t_{mx}} = \{t_1, t_2, t_3, \cdots, t_{N_b}\}$

So that

- Life-cycle network maintenance cost C_{mx} (calculated based on Eqn. 6) is minimized
- Life-cycle risk of the bridge network R_{lfc} (calculated based on Eqn. 14) is minimized

Subjected to

- Time of the maintenance t_{jj} ($jj = 1, 2, \ldots, N_b$) $\leq T_{jj} - 10$ years

T_{jj} is the target service life of the jjth bridge (considered as 75 years herein).

Information about the bridge network investigated

The investigated bridge network is located in Chester County, PA. It consists of four prestressed concrete bridges and six steel bridges. The configuration of the bridge network is shown in Figure 1. Two links in opposite directions connecting the same two nodes constitute one segment. Basic information on all of the 10 bridges (named as B1, B2, …, B10) and all the links (named as L1, L2,…, L22) is given in Tables 1 and 2, respectively. Three pairs of bridges exist in this network, which are bridges B2/B3, B6/B7, and B9/B10. The two bridges in a pair carry the two-way traffic in opposite directions with the same or similar geometry as well as identical construction/reconstruction time. The intended service life of each bridge is considered as 75 years. Existing service lives from the time of construction/reconstruction until year 2020 of all 10 bridges are considered, as corrosion damage has already been in place on these bridges at year 2020. The time span of this bridge network is considered as 45 years, which is the remaining service life of the oldest bridge in the bridge network (bridge B5).

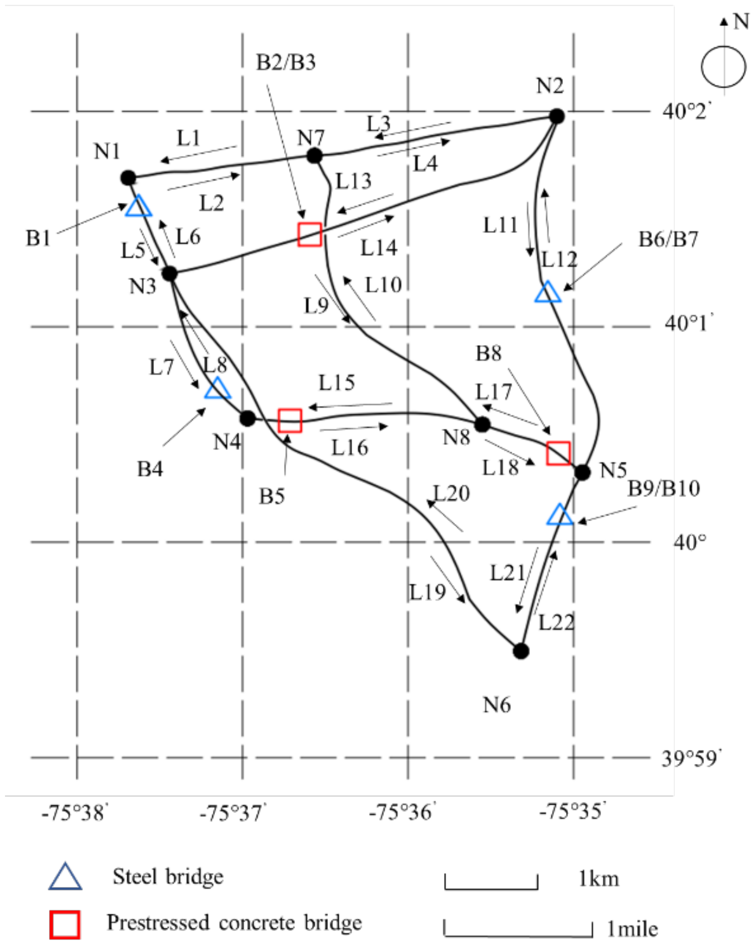


Figure 1. Bridge network in Chester County, PA.

Table 1. Information on bridges in the network.

Bridge ID	Structural Number in NBI Database	Latitude (degree)	Longitude (degree)	Bridge	Material of Super-structure	Length (m)	Width (m)	Year Built	Time of Last Renovation	Years in Service before 2020
B1	10066	40.02769	-75.6278	MGS	S	9.5	38.7	1954	Reconstructed at year 2000	20
B2	10003	40.02414	-75.6089	MGS	PC	27.9	13.3	1994	-	26
B3	10001	40.02392	-75.6090	MGS	PC	27.9	13.3	1994	-	26
B4	10060	40.01152	-75.6156	MGS	S	32.3	9.8	1968	Reconstructed at year 2009	11
B5	10402	40.00949	-75.6144	BGS	PC	28.0	13.9	1968	1990	30
B6	10112	40.01926	-75.5861	MGC	S	35.7	13.4	1968	1998	22
B7	10111	40.01936	-75.5859	MGC	S	35.7	13.4	1968	1998	22
B8	10403	40.00548	-75.5824	BGS	PC	17.7	13.4	1968	2000	20
B9	10109	40.00163	-75.5848	MGS	S	15.1	14.7	1968	1998	22
B10	10108	40.00177	-75.5844	MGS	S	15.1	13.4	1968	1998	22

Note: MGS means multi-girder simply supported, MGC means multi-girder continuous, BGS means box girder simply supported, S means steel, PC means prestressed concrete.

Table 2. Information on links in the network.

Link Number	First Node	Second Node	Free Travel Time(min)	Length of Link (km)	Critical Capacity (cars/h)	Number of Lanes	Free Speed (km/h)
1(2)	1	7	1.3	1.60	2000	1	72
3(4)	2	7	1.8	2.12	2000	1	72
5(6)	1	3	0.7	0.89	8000	4	72
7(8)	3	4	0.6	0.75	4000	2	72
9(10)	7	8	3.2	2.96	2000	1	56
11(12)	2	5	2.2	3.31	4000	2	90
13(14)	2	3	2.4	3.69	4000	2	90
15(16)	4	8	3.1	2.91	2000	1	56
17(18)	5	8	1.2	0.82	2000	1	56
19(20)	3	6	2.8	4.24	4000	2	90
21(22)	5	6	1.4	2.09	4000	2	90

Note: Free speed is the speed of vehicles traveling if there were no congestion or other adverse conditions.

A typical cross section of a steel girder is shown in Figure 2. The notation of each geometrical parameter and its value associated with the cross section in Figure 2 for different bridges are presented in Table 3. Typical cross sections of the prestressed concrete girders associated with four prestressed concrete bridges (B2, B3, B5, and B8) are shown in Figure 3. Each prestressed strand consists of seven wires. The diameter of strands for the bridge pair B2/B3 is 12.7 mm while the diameter is 11.11 mm for bridges B5 and B8.

For steel bridges in the bridge network, the corrosion models considered for the steel bridge in Montgomery County were applied. A pitting corrosion model of prestressed steel reinforcements was considered for concrete bridges in the bridge network. Detailed information on the pitting corrosion model is provided in Stewart (2004). The time-variant distribution of flexural/shear girder resistances is obtained through Monte Carlo simulation, where samples of random variables related to structural resistances are generated and used into the expression of resistances of steel/concrete girders according to AASHTO (2017). Detailed information on these random variables are shown in Table 4. Normal distribution is used to represent time-variant resistance of girders based on the results of goodness-of-fit test.

The nominal load effects are calculated based on HL-93 loads in AASHTO (2017). The performance function of the interior girders of the bridges in the bridge network can be generalized as

$$g(t) = \gamma_R R(t) - Q_{steel}\lambda_{steel} - Q_{conc}\lambda_{conc} - Q_{conc,w}\lambda_{conc,w} - Q_{misc}\lambda_{misc} - 0.85 \cdot Q_{ll} \cdot D \cdot \lambda_{ll} \cdot \lambda_D \cdot (1 + I_g) \quad (15)$$

where $R(t)$ is the time-variant resistance; γ_R is the model error of resistance; Q_{steel}, Q_{conc}, $Q_{conc,w}$ and Q_{misc} are the nominal dead loads induced by steel structures, concrete deck, concrete wearing surface, and miscellaneous items (such as utilities), respectively; λ_{steel}, λ_{conc}, $\lambda_{conc,w}$ and λ_{misc} are the uncertainty factors associated with those dead loads; Q_{ll} and D are the nominal live load effect induced by HL-93 and nominal girder distribution factor, respectively; λ_{ll} and λ_D are the model errors of Q_{ll} and D, respectively; I_g is the dynamic load allowance.

The information on the nominal values in Equation 15 associated with all 10 bridges is summarized in Table 5 and the information on the model errors and uncertainty factors in Equation 15 associated with all 10 bridges is given in Table 6.

The uncertainty factors associated with dead load and live load effects are provided in Han *et al.* (2021b). First Order Reliability Method (FORM) (Ang & Tang 1984) is adopted to obtain time-variant reliability profiles associated with individual failure modes of girders. It is assumed herein that the failure of two adjacent girders results in the failure of the bridge superstructure. For one steel girder, flexural failure mode and shear failure mode are in series.

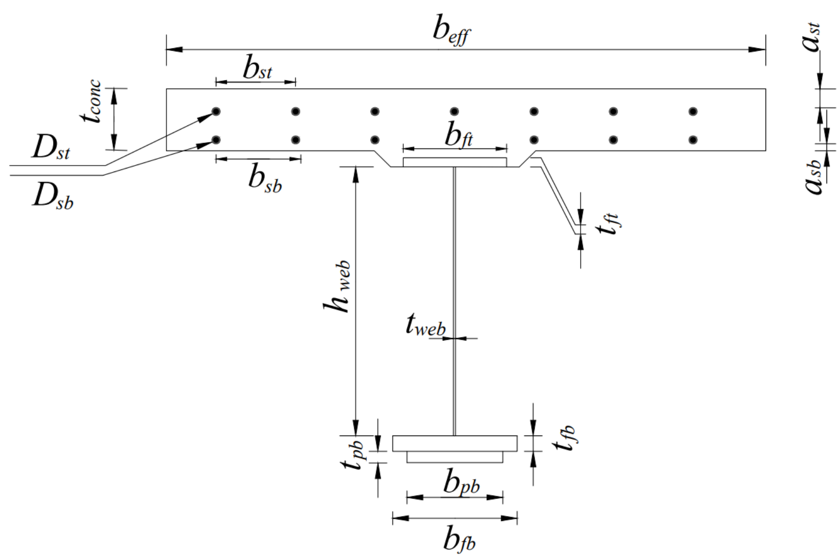


Figure 2. Typical cross section of a composite girder in steel bridges.

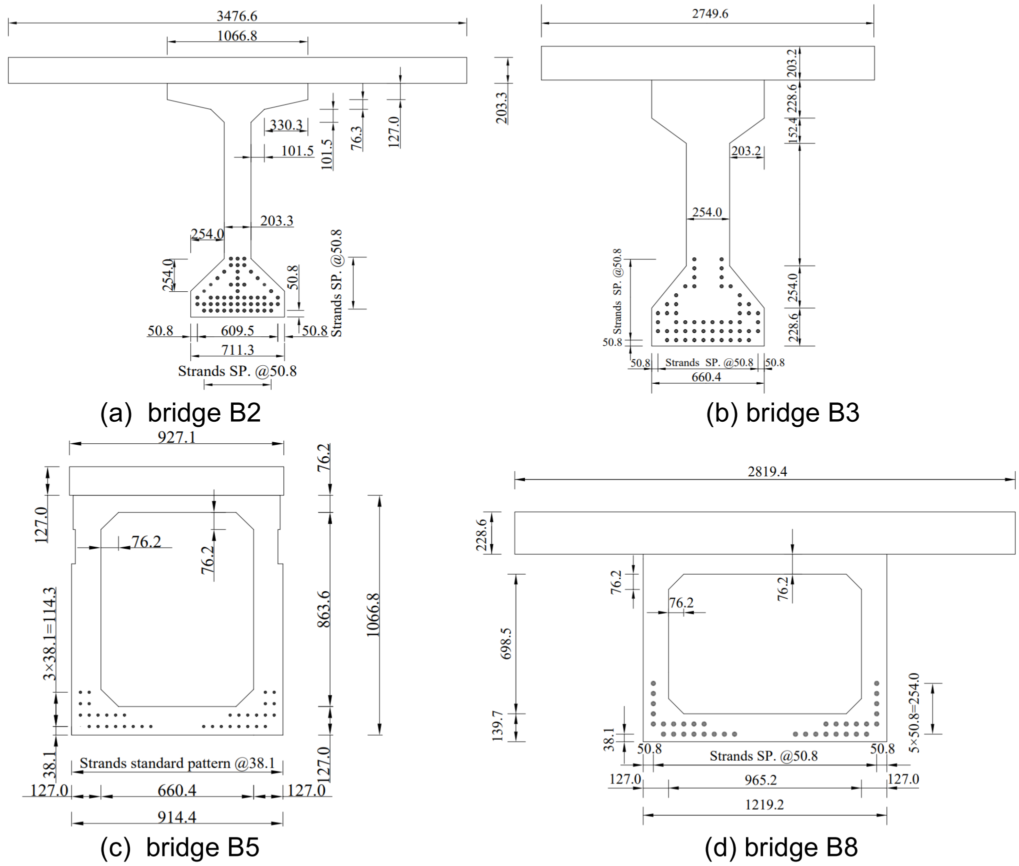


Figure 3. Cross sections of prestressed concrete bridges (dimensions are in mm).

Table 3. Geometric parameters in Figure 2 (all dimensions are in mm).

Parameter	Description	B1	B4	B6 and B7 Flex	B6 and B7 Shear	B9	B10
b_{eff}	Effective tributary width of the deck	1,397.0	2,133.6	N/A	N/A	2,209.8	1,930.4
t_{conc}	Depth of concrete deck	203.2	203.2	N/A	N/A	203.2	203.2
b_{st}	Top reinforcement spacing	279.4	304.8	N/A	N/A	315.7	321.8
b_{sb}	Bottom reinforcement spacing	279.4	228.6	N/A	N/A	203.2	254.0
a_{st}	Top concrete cover depth	69.9	66.7	N/A	N/A	66.7	66.7
a_{sb}	Bottom concrete cover depth	69.9	41.3	N/A	N/A	41.3	41.4
D_{st}	Diameter of top reinforcement	15.9	12.7	N/A	N/A	12.7	12.7
D_{sb}	Diameter of bottom reinforcement	15.9	15.9	N/A	N/A	15.9	15.9
b_{ft}	Top flange width	228.2	406.4	253.7	253.7	253.1	253.1
t_{ft}	Top flange thickness	17.3	28.6	19.0	28.5	16.2	16.2
h_{web}	Web height	572.4	1,295.4	683.5	683.5	645.6	645.6
t_{web}	Web thickness	11.2	9.5	12.4	12.4	11.8	11.8
b_{fb}	Bottom flange width	228.2	457.2	253.7	253.7	253.1	253.1
t_{fb}	Bottom flange thickness	17.3	50.8	19.0	28.5	16.2	16.2
b_{pb}	Bottom cover plate width	N/A	N/A	N/A	N/A	355.6	355.6
t_{pb}	Bottom cover plate thickness	N/A	N/A	N/A	N/A	22.2	15.9

Note: For non-composite bridges B6 and B7, the resistance of concrete is not considered. Only bridges B9 and B10 have bottom cover plates. N/A means not applicable.

Table 4. Random variables used in the girder capacity calculations.

Random Variable	Notation	Mean	Coefficient of Variation	Distribution
Surface chloride content at deck (kg/m^3)[a]	C_0	3.5	0.5	Lognormal
Critical chloride content (kg/m^3)[a]	C_{cr}	0.9	0.19	Uniform[j]
Corrosion current density ($\mu A/cm^2$)[a]	i_{corr}	1.5	0.33	Uniform[k]
Chloride diffusion coefficient (mm^2/year)[a, b]	D_c	122.68	0.75	Lognormal
Pitting factor[a]	R_{pit}	3	0.33	Normal[a,l]
Height of concrete slab (mm)[c]	h_{slab}	$h_{slab,nom} + 0.8$[h]	$3.6/h_{slab,mean}$[i]	Normal[m]
Bottom cover of transverse rebar in concrete (mm)[c]	C_{bot}	$C_{bot,nom} + 8.6$[h]	$14.7/C_{top,mean}$[i]	Normal[m]
Top cover of transverse rebar in concrete (mm)[c]	C_{top}	$C_{top,nom} + 19.8$[h]	$16.5/C_{top,mean}$[i]	Normal[m]
Distance from extreme compression fiber to the centroid of prestressing strands (mm)[d]	d_p	$d_{p,nom}$[h]	0.02	Lognormal
Yield strength of non-prestressed Grade 40 steel rebar (used in Bridge B1) (MPa)[e]	f_{yr1}	312.1	0.116	Normal[m]
Yield strength of non-prestressed Grade 60 steel rebar (used in Bridges B4, B9, and B10) (MPa)[e]	f_{yr2}	465.1	0.098	Normal[m]
Tensile strength of prestressing steel (MPa)[d]	f_{pu}	1396.1	0.025	Lognormal
Compressive strength of concrete (MPa)[e]	f_c	23.37	0.18	Normal[m]
Yield strength of carbon steel[f]	F_y	238.7	0.065	Normal[m]
Modulus of elasticity of steel[f]	E	2×10^5	0.019	Normal[m]
Corrosion rate of carbon steel (mm/year)[g]	r_{corr}	0.067	1	Exponential
Corrosion rate ratio of A709-50CR to carbon steel[g]	r_{ss}	0.014	0.378	Uniform[n]
Coating life (year)[g]	t_0	15.4	0.344	Lognormal

Note: (a) Based on Stewart and Rosowsky (1998); (b) Based on Attard and Stewart (1999); (c) Based on Mirza and Macgregor (1979), the uncertainty of concrete cover is only considered for non-prestressing reinforcements; (d) Based on Akgül and Frangopol (2004); (e) Based on Macgregor *et al.* (1983); (f) Strength of carbon steel plates is based on Kennedy and Gad Aly (1980); (g) Based on Han *et al.* (2021b); (h) X_{nom} refers to the nominal value of X; (i) X_{mean} refers to the mean of X; (j) The associated lower and upper bound of this uniform distribution is 0.6 and 1.2 kg/m^3, respectively; (k) The associated lower and upper bound of this uniform distribution is 1 and 2 $\mu A/cm^2$, respectively; (l) The distribution of R_{pit} is truncated at 1; (m) Normal distribution is assumed; (n) The associated lower bound and upper bound are 0.0011 and 0.0534, respectively.

Table 5. Nominal values of load effects in Equation 15.

Failure mode	Q_{steel}	Q_{conc}	$Q_{conc,w}$	Q_{misc}	Q_{ll}	D
B1 flexural	25.75	75.61	22.62	14.03	566.91	0.468
B1 shear	4.98	31.76	5.92	5.56	268.84	0.565
B2 flexural	0	3414.16	365.19	270.16	2759.64	0.750
B3 flexural	0	3023.96	383.43	286.22	2759.64	0.761
B4 flexural	408.13	1333.78	400.14	398.23	3428.07	0.533
B4 shear	50.48	165.02	49.51	49.28	442.93	0.882
B5 flexural	0	1288.06	130.90	107.65	2785.34	0.146
B6 flexural	26.96	93.4182	26.39	47.73	560.46	0.611
B6 shear	23.04	79.75	22.51	40.74	320.88	0.779
B7 flexural	26.96	93.4182	26.39	47.73	560.46	0.611
B7 shear	23.04	79.75	22.51	40.74	320.88	0.779
B8 flexural	0	954.64	159.68	306.68	1410.83	0.701
B9 flexural	97.07	303.32	91.21	212.95	1100.82	0.569
B9 shear	25.62	80.11	24.06	56.22	330.75	0.895
B10 flexural	86.61	264.83	79.49	186.17	1100.82	0.513
B10 shear	22.86	69.92	20.99	49.15	330.75	0.817

Note: The units for load effects associated with shear and flexural failure modes are kN and KNm, respectively.

Table 6. Model errors and uncertainty factors in Equation 15.

Random Variable	Notation	Mean	Coefficient of Variation	Distribution
Model error of the resistance of composite girder[a]	γ_{mg}	1.05	0.06	Normal[e]
Uncertainty factor associated with the weight of steel[a]	λ_{steel}	1.03	0.08	Normal[e]
Uncertainty factor associated with the weight of the concrete[a]	λ_{conc}	1.05	0.10	Normal[e]
Uncertainty factor associated with the wearing surface of the concrete[a]	$\lambda_{conc,w}$	1[d]	0.25	Normal[e]
Uncertainty factor associated with the miscellaneous items[a]	λ_{misc}	1.03	0.08	Normal[e]
Uncertainty factor associated with annual maximum moment induced by HL-93[a]	λ_{Ml}	1.20	0.19	Gumbel
Uncertainty factor associated with annual maximum shear induced by HL-93[a]	λ_{Vl}	1.18	0.19	Gumbel
Model error associated with GDFs of flexural live load effects[b]	λ_D	0.73	0.146	Triangular[f]
Model error associated with GDFs of shear live load effects[c]	$\lambda_{Ds,int}$	0.886	0.060	Uniform[g]

Note: (a) Based on Nowak (1999); (b) Based on Kim and Nowak (1997), Nowak *et al.* (1998), and Eom and Nowak (2001); (c) Based on Suksawang *et al.* (2013); (d) The mean value of $\lambda_{conc,w}$ is assumed to be 1; (e) Normal distribution is assumed; (f) The associated lower bound, mode, and upper bound are 0.50, 0.68, and 1.01, respectively; (g) The associated lower and upper bound are 0.794 and 0.977, respectively.

Information on the replacement cost of the superstructure and the replacement time is given in Table 7. The replacement cost of the entire structure of each bridge is estimated as 1.40 times that of the replacement cost of its superstructure (Saito *et al.* 1988). The repainting cost for the superstructure of a steel bridge is estimated based on a 15% repainting-replacement cost ratio (FHWA 2011).

Two cases are investigated to determine the influence of user equilibrium approach on the user cost, namely

• Case I: Deterministic user equilibrium (All-or-nothing assignment is used)

• Case II: Stochastic user equilibrium (Logit assignment with $\theta = 1$); $\theta = 1$ in Case II indicates that travelers are moderately sensitive to the path cost.

The daily traffic volumes of cars and trucks on each link are given in Table 8 (PennDOT 2021). A total of 500 paths are identified for 56 O-D pairs using Yen's algorithm (Yen 1971). O-D demand maxtrix is determined using the OD demand estimator in Van Zuylen and Willumsen (1980). Information on daily O-D demand of cars and trucks is given in Tables 9 and 10, respectively.

For steel bridges, girder replacement can be carried out using either carbon steel girders or A709-50CR girders. The cost premium of A709-50CR in Kogler (2015) is adopted herein to estimate the replacement cost of the superstructure of steel bridges using A709-50CR girders. Detailed information on replacement cost of steel bridges using A709-50CR is shown in Table 11. Monetary discount rate rr is 2%. An operational constraint is added that a pair of bridges needs to be replaced during the same replacement actions.

Table 7. Replacement cost of bridge superstructure and replacement duration.

Bridge	B1	B2	B3	B4	B5	B6	B7	B8	B9	B10
Rebuilding Cost of Superstructure (10^5 USD)	8.54	6.47	6.47	7.31	6.81	11.09	11.09	4.15	5.17	4.71
Rebuilding Duration (days)	122	113	113	117	114	130	130	100	106	103

Source: Estes and Frangopol (1999), Frangopol and Liu (2007)

Table 8. Daily Traffic flow on each link.

Link Number	Car	Truck
1	7,274	460
2	7,661	456
3	3,960	262
4	4,010	260
5	20,254	1,579
6	22,560	1,176
7	8,742	476
8	7,190	367
9	3,069	94
10	3,017	94
11	13,630	1,561
12	13,146	1,654
13	14,372	2,163
14	13,430	2,295
15	4,631	142
16	4,623	144
17	6,235	542
18	6,277	546
19	13,980	2,486
20	14,449	1,997
21	20,467	587
22	19,919	661

Source: PennDOT (2021)

Table 9. Information on daily O-D demand of cars.

Origin Node \ Destination Node	N1	N2	N3	N4	N5	N6	N7	N8
N1	0	0	2,038	934	1,292	10,141	2	4,806
N2	0	0	8,111	3,148	1	12,662	0	0
N3	3,882	8,675	0	0	3,922	5	920	658
N4	964	1,903	0	0	417	4	227	215
N5	529	1	3,767	669	0	4,834	0	0
N6	10,916	12,460	4	5	3,990	0	1	1,843
N7	1	0	588	266	0	5	0	1
N8	4,854	0	550	272	0	1,803	1	0

Table 10. Information on daily O-D demand of trucks.

Origin Node \ Destination Node	N1	N2	N3	N4	N5	N6	N7	N8
N1	0	0	59	20	0	1,986	0	0
N2	0	0	1,482	263	1,139	255	0	0
N3	4	1,522	0	0	10	1,523	0	0
N4	1	179	0	0	0	74	0	0
N5	0	1,117	90	0	0	0	0	305
N6	816	464	1,170	77	0	0	0	10
N7	3	0	0	0	0	0	0	0
N8	0	1	0	0	312	5	0	0

Table 11. Replacement cost of steel bridge superstructure using A709-50CR.

Bridge	B1	B4	B6	B7	B9	B10
Rebuilding cost of superstructure (10^5 USD)	9.34	7.90	11.95	11.95	5.66	5.16

Source: Estes and Frangopol (1999), Kogler (2015)

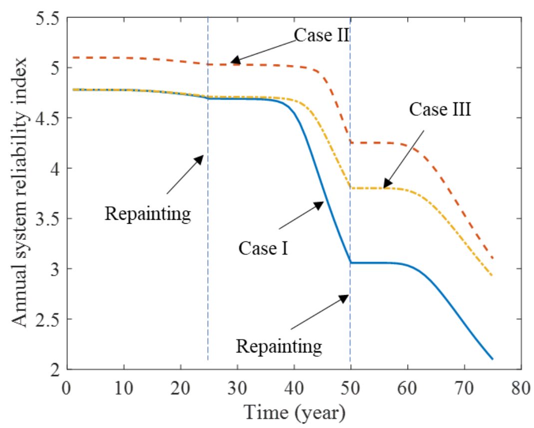
CHAPTER 3

Findings

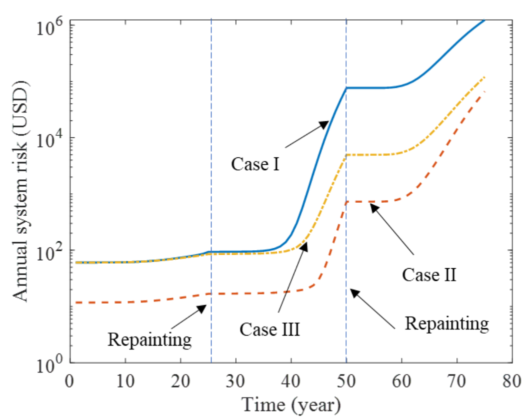
RESULTS OF PARAMETRIC ANALYSIS OF INFLUENCE FACTORS ON COST-EFFECTIVENESS OF A709-50CR

Time-variant system reliability/risk profiles of the steel bridge in Montgomery County, PA

Time-variant system reliability/risk profiles of the steel bridge in Montgomery County, PA, associated with Cases I through III and system Model II, are shown in Figure 4. It can be seen that a high correlation among girder resistances (i.e., Case I) will have a negative impact on the reliability/risk of the series-parallel system associated with the failure of any two adjacent girders. The annual reliability and risk profiles associated with Case III are bounded by those associated with Cases I and II.



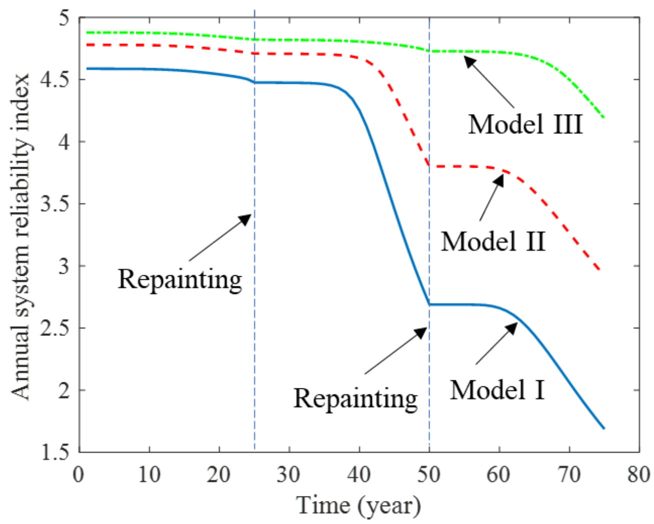
(a)



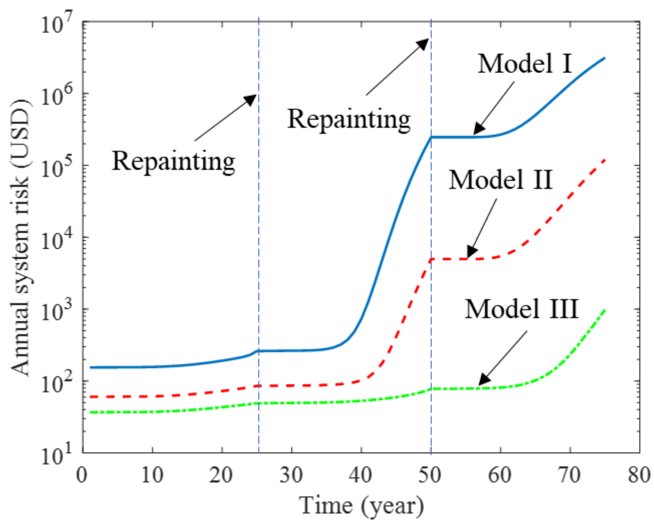
(b)

Figure 4. Time-variant annual (a) system reliability index profiles and (b) system risk profiles associated with correlation Cases I, II, and III.

Time-variant system reliability/risk profiles of the steel bridge in Montgomery County, PA, associated with system Models I through III and correlation Case III, are shown in Figure 5. It can be seen that the series model is the most critical system for reliability/risk evaluation. System reliability becomes higher with an increase in the number of girders involved in a sub-parallel system (which indicates a higher redundancy).



(a)

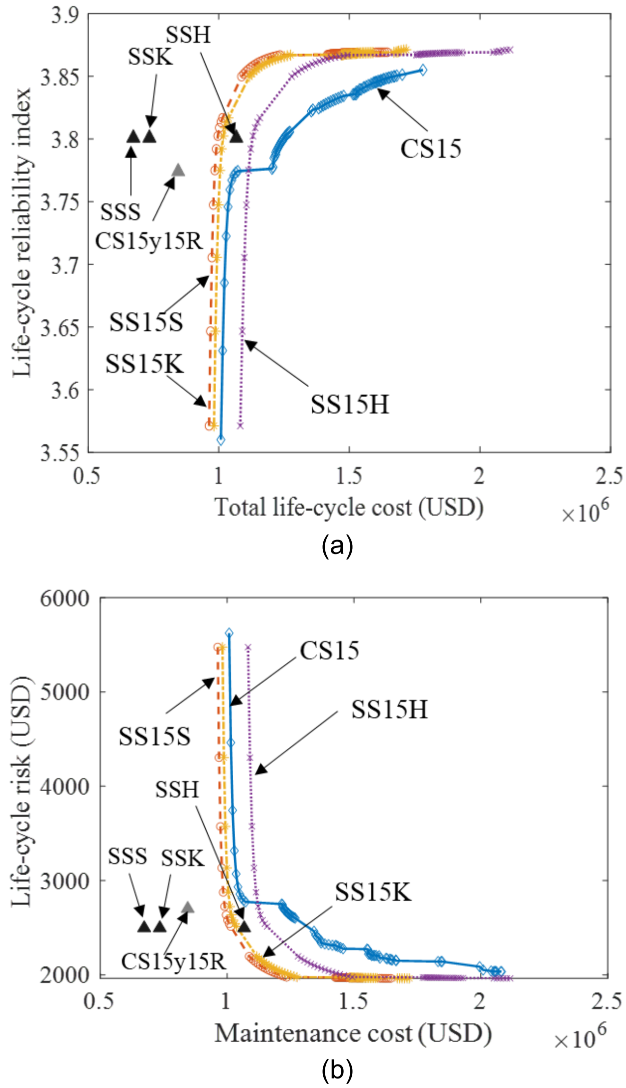


(b)

Figure 5. Time-variant annual (a) system reliability index profile, and (b) system risk profile associated with three different system models based on correlation Case III.

Optimization results

Reliability- and risk-based optimization results associated with correlation Case III and system Model II are shown in Figure 6. The comparison between Pareto front of carbon steel (CS15) and three Pareto fronts of A709-50CR (SS15S, SS15K, and SS15H) in Figure 6 can prove the advantage of using A709-50CR in girder replacement, i.e. the life-cycle cost can be lessened in cases of low-cost (SS15S) and medium-cost (SS15K) premiums. Even in the case of high-cost (SS15H) premium, the advantage of using A709-50CR girder for replacement still exists since only using A709-50CR can achieve a high reliability or low risk level (e.g., when the life-cycle reliability index is required to be higher than 3.8 or the life-cycle risk is required to be lower than 2750 USD). If such stringent reliability or risk requirement can be relaxed, repainting at every 15 years (solution CS15y15R) and building the bridge with A709-50CR (solutions SSS, SSK, and SSH) are both cost-effective solutions. In this case, the cost-effectiveness of building an A709-50CR bridge compared with frequent repainting hinges upon the cost-premium of A709-50CR over carbon steel. After checking the detailed maintenance plan associated with the solutions in the Pareto front, it is determined that the difference between the maintenance schedules associated with reliability-based maintenance strategy and risk-based maintenance strategy under a fixed maintenance budget is minimal. This is due to the inconsequential effect of monetary discount rate on optimal solutions.



Note: In the designation of Pareto fronts, “CS” and “SS” for case notation refer to the carbon steel and A709-50CR, respectively; “15” refers to the cost ratio of repainting to replacement , i.e. 15%; “S”, “K”, and “H” represents low, medium, and high cost premiums, respectively; grey triangle solution “CS15y15R” refers to applying repainting every 15 years on the carbon steel bridge with 15% repainting/replacement cost ratio ; Dark star solutions “SSS”, “SSK,” and “SSH” refer to building an A709-50CR bridge with cost premiums being “S”, “K”, and “H”.

Figure 6. Optimal Pareto fronts associated with 15% repainting/replacement cost ratio, correlation case III and system model II (a) reliability-based optimization, (b) risk-based optimization.

RESULTS OF COST-EFFECTIVENESS ANALYSIS OF A709-50CR ON A BRIDGE NETWORK LEVEL

Traffic flow and user cost in user equilibrium states

Five categories of total daily travel time on each segment (i.e., the road connecting two nodes) of the bridge network in Chester County, PA, are set up herein: less than 300 hours, less than 500 hours, less than 1,000 hours, less than 1,500 hours, and more than 1,500 hours. The total daily travel time on each segment associated with original state (i.e., no bridge failure occurs) and the state in which bridge B1 failed is plotted in Figure 7. The associated detailed information on the daily travel time on each segment is shown in Table 12. It can be seen that the adopted user equilibrium type can make a difference in the estimated total travel time on each segment for both the original network configuration and the network configuration associated with failure of bridge B1. Total travel time on each segment in Cases I and II does not differ significantly when no bridge failure occurs. However, the difference in total travel time on each segment for these two cases can be observed with clarity when bridge B1 fails. The total travel time on some segments in Case II is larger than that in Case I (shown in Figure 7(b) and 7(d); see also segments S7 and S8 for Case II in Table 12), whereas the total travel time on some other segments in Case II is smaller than that in Case I (shown in Figure 7(b) and 7(d); see also segments S6 and S11 for Case II in Table 12). Another major observation is that significant change of the total travel time on segments due to bridge failure associated with deterministic assignment (i.e., Case I) and stochastic assignment with moderate path cost sensitivity (i.e., Case II) (see Figure 7(a), 7(b), 7(c), and 7(d); see also segments S1, S2, S6, S8, S10, and S11 for Case I, and segments S1, S2, S5, S7, S10, and S11 for Case II in Table 12).

Information on user costs associated with each bridge failure is presented in Table 13, where the paradox indicated by Sheffi and Daganzo (1978) can be observed for the failure of bridge B5 in Case II. In Case I where deterministic traffic assignment is adopted, the user cost due to either extra travel time or distance is always positive, as travelers are forced to use higher cost paths when bridge failure occurs. In Case II, as users may use high-cost path when facing multiple path choices, the removal of certain links leads to a reduction in the number of paths available between O-D pairs, which in turn leads to a decrease in the possibility that a high-cost path is chosen. Therefore, the failure of some bridges may counterintuitively reduce the total travel time or distance, rendering negative user cost values. For the failure of the same bridge, the user cost associated with Case II is lower than that associated with Case I.

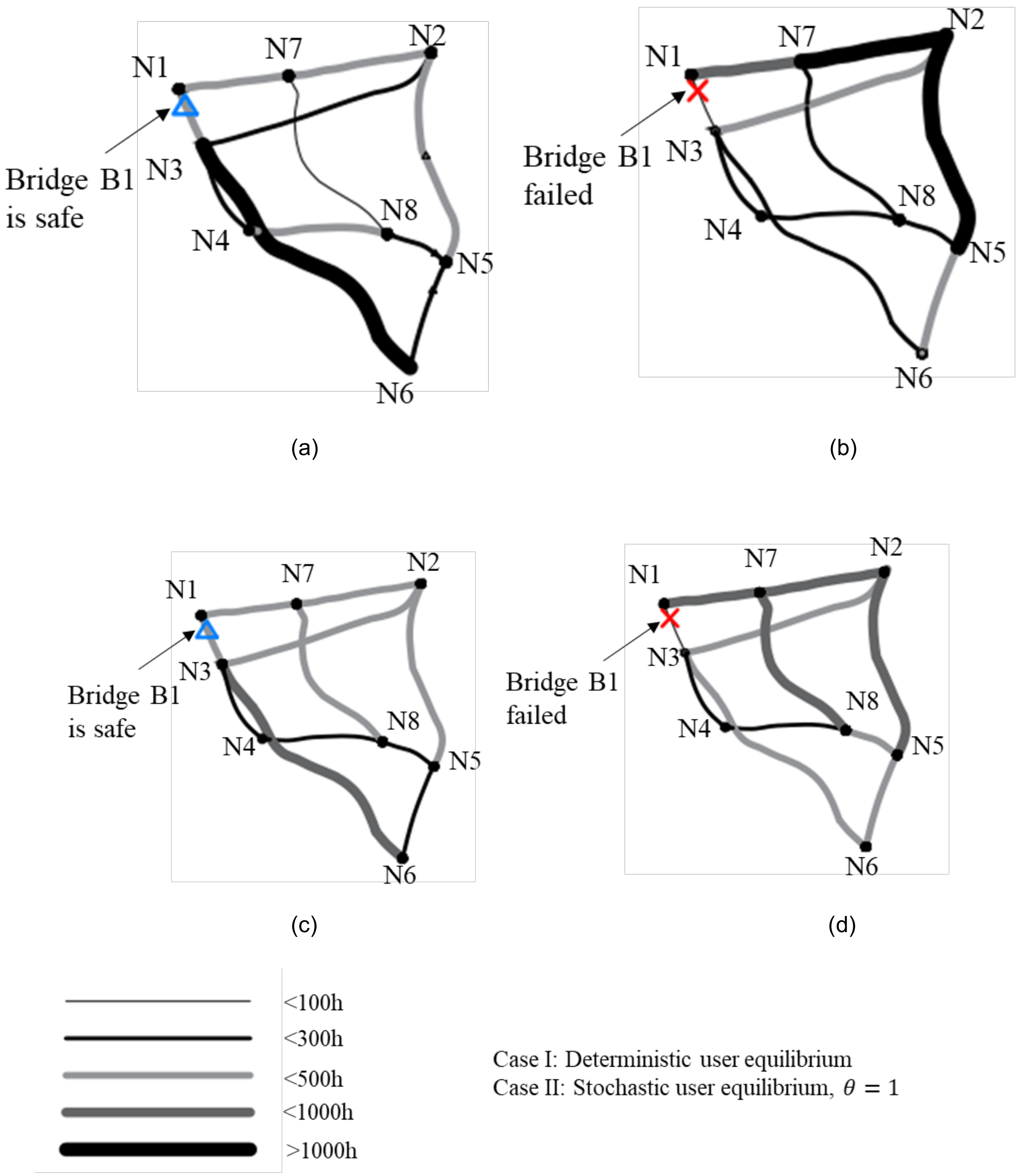


Figure 7. Total daily travel time on segments: (a) Case I, original; (b) Case I after failure of bridge B1; (c) Case II, original; (d) Case II after failure of bridge B1.

Table 12. Daily travel time on segments of the network (hours).

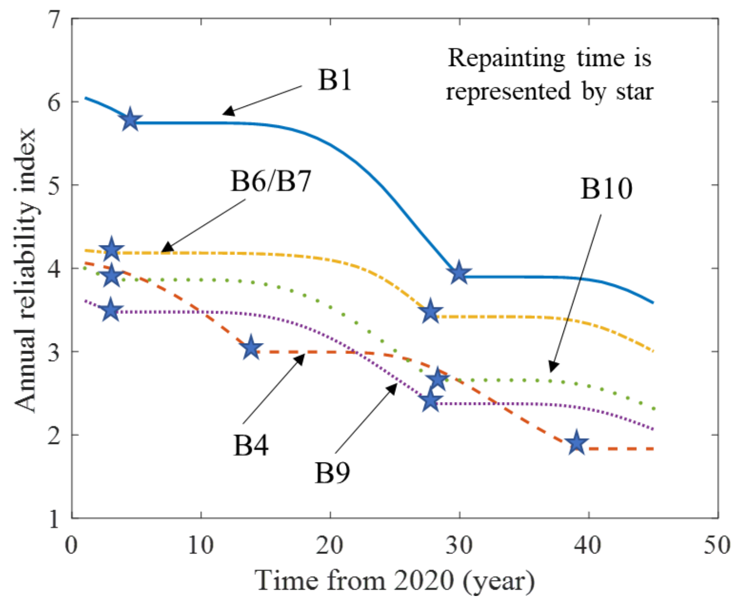
Segment Number	Associated Nodes	Case I Intact	Case I After Bridge B1 Fails	Case II Intact	Case II After Bridge B1 Fails
S1	N1, N7	43.6	963.2	339.1	963.2
S2	N2, N7	0.0	1,064.3	287.1	759.5
S3	N1, N3	517.5	0	523.1	0
S4	N3, N4	189.4	92.7	168.2	124.8
S5	N7, N8	0.1	515.7	330.3	1,107.7
S6	N2, N5	1,031.0	1,953.7	1,084.4	1,483.2
S7	N2, N3	1,011.7	1,409.6	1,150.9	1,706.6
S8	N4, N8	643.0	143.7	496.3	332.6
S9	N5, N8	80.5	80.5	203.2	324.3
S10	N3, N6	1,656.2	497.0	1,536.1	654.3
S11	N5, N6	1,124.1	1,646.9	977.5	1,402.5

Table 13. Summary of failure consequence associated with daily user cost (USD).

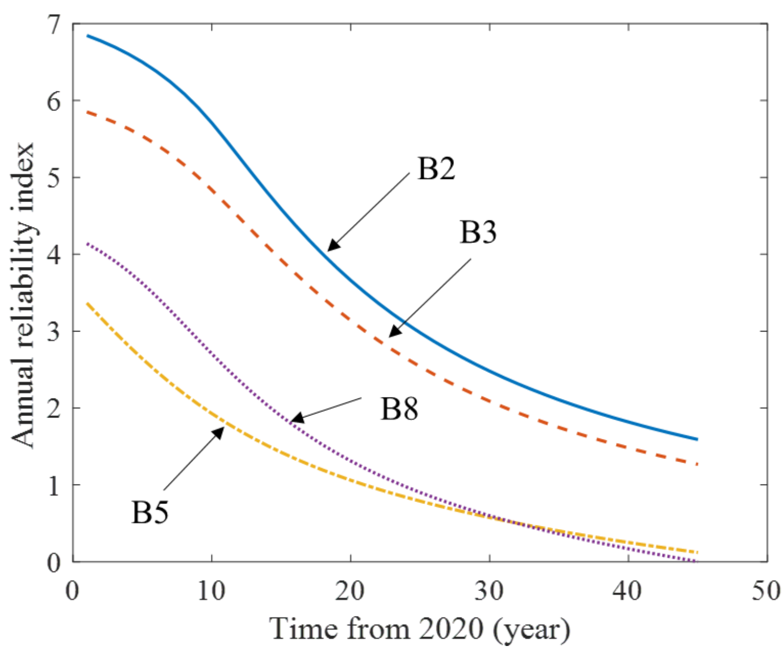
Bridge Failure	Case I Extra Travel Time	Case I Extra Travel Distance	Case II Extra Travel Time	Case II Extra Travel Distance
B1	7.06×10^4	1.44×10^4	6.02×10^4	1.09×10^4
B2	1.05×10^4	1.43×10^3	7.09×10^3	6.34×10^2
B3	9.88×10^3	1.37×10^3	6.39×10^3	4.83×10^2
B4	8.58×10^3	1.25×10^3	7.45×10^3	1.34×10^3
B5	2.83×10^3	8.30×10^2	-8.57×10^1	7.38×10^2
B6	1.48×10^4	3.91×10^3	1.19×10^4	3.08×10^3
B7	1.47×10^4	4.05×10^3	1.19×10^4	3.24×10^3
B8	1.14×10^4	3.53×10^3	3.53×10^3	2.62×10^3
B9	3.63×10^4	5.40×10^3	3.09×10^4	5.19×10^3
B10	3.30×10^4	5.26×10^3	2.78×10^4	4.87×10^3

Time-variant reliability and network risk profiles

Time-variant annual reliability index profiles of all 10 bridges in the bridge network are shown in Figure 8. It can be seen that the safety level of steel bridges in the bridge network (Figure 8(a)) is higher than that of prestressed concrete bridges (Figure 8(b)) due to the repainting actions (time of repainting actions is represented by a star in Figure 8(a)).



(a) steel bridges



(b) prestressed concrete bridges

Figure 8. Annual reliability profiles of individual bridges (time of repainting of steel bridges is marked by stars).

Time-variant risk profiles associated with Cases I and II are plotted in Figure 9. It can be seen that as user cost estimation using deterministic user equilibrium approach (i.e., Case I) leads to an overestimation of failure consequence, an overestimation of risk will occur if the deterministic user equilibrium approach is used.

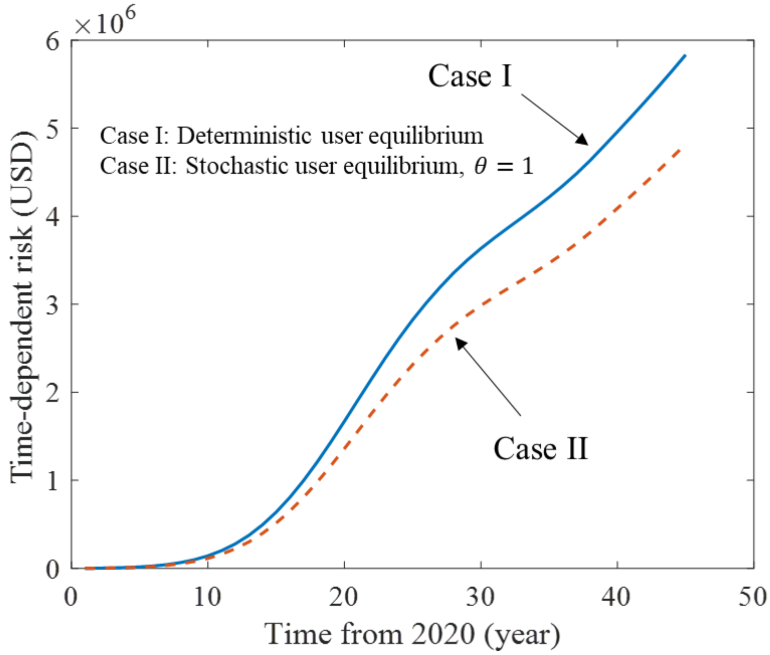
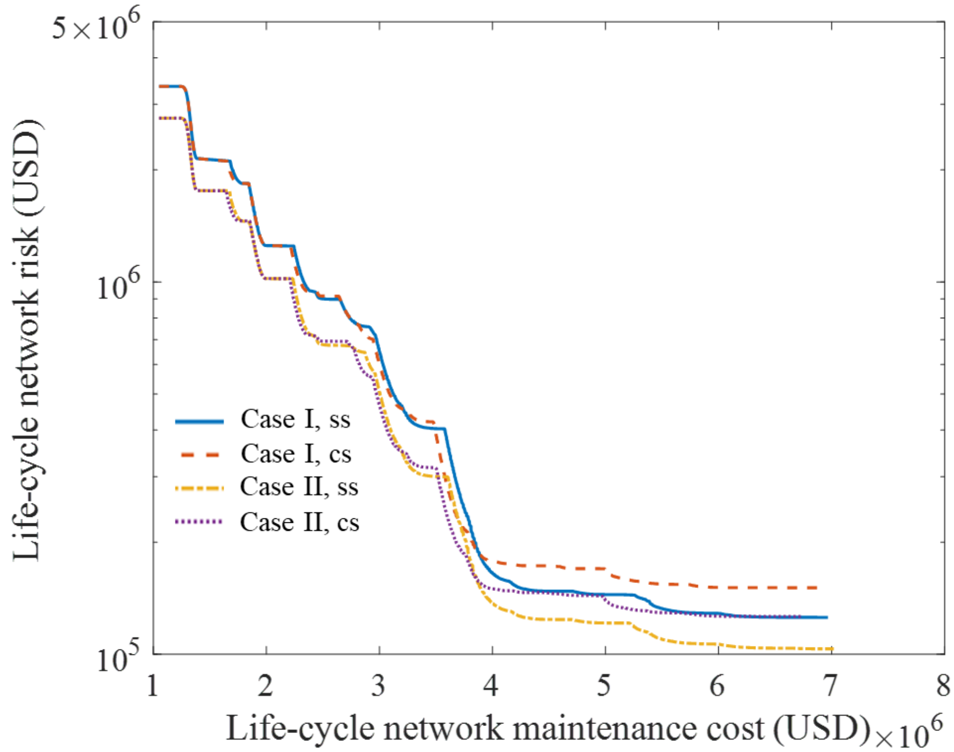


Figure 9. Time-dependent risk profiles.

Optimization results for risk-based life-cycle management of bridge network

The optimal Pareto fronts associated with life-cycle management of the bridge network in Chester County, PA, are shown in Figure 10. The two Pareto fronts associated with Case I are termed as "Case I, ss" and "Case I, cs", where "ss" and "cs" refers to A709-50CR steel and carbon steel girders, respectively. Similarly, the two Pareto fronts associated with Case II are referred to as "Case II, ss" and "Case II, cs". Taking Case I as an example, for the two Pareto fronts associated with the same type of user equilibrium, when the target life-cycle risk is low (smaller than 2.27×10^5 USD in this case), using A709-50CR steel to perform replacement leads to a lower cost than using carbon steel. When the target life-cycle risk is higher, the Pareto front of "Case I, ss" overlaps that of "Case II, cs" at several risk level ranges (from 1.23×10^6 USD to 1.59×10^6 USD, from 2.14×10^6 USD to 2.56×10^6 USD, from 3.16×10^6 USD to 4.44×10^6 USD), which indicates that if the target risk falls in these ranges, the cost-effectiveness of using A709-50CR or carbon steel is the same. Comparison between the two Pareto fronts associated with using the same type of materials shows that due to an overestimation of the life-cycle risk by using the deterministic user equilibrium approach, more costly maintenance plans may be requested under a fixed target life-cycle network risk.



Note: Case I and Case II are associated with deterministic user equilibrium and stochastic user equilibrium, respectively. “ss” refers to A709-50CR steel; “cs” refers to carbon steel.

Figure 10. Risk-based optimal Pareto fronts.

CHAPTER 4

Recommendations

COST-EFFECTIVENESS OF A709-50CR ON AN INDIVIDUAL BRIDGE LEVEL

After evaluating the cost-effectivness of multiple types of maintenance actions on a multi-girder steel bridge in Montgomery County, PA, the conclusions and recommendations on the cost-effectiveness of A709-50CR at an individual bridge level are drawn as follows:

- Time-variant correlation among girder resistances can have a significant impact on system reliability and risk. For a series or series-parallel system to model bridge superstructure, high correlation among girder resistances is more critical than low correlation. This suggests that a larger budget may be allocated for a bridge with highly correlated resistance among different girders compared to a bridge with low resistance correlation.
- System models have a profound influence on system reliability and risk profiles. For multi-girder bridges, system failure is governed by the failure of a number of adjacent girders. The system of which the failure is associated with a larger number of adjacent girders has a higher redundancy. A more critical system (e.g., series system) requires a larger maintenance budget to achieve a specific reliability/risk level compared with a more redundant system.
- A709-50CR prevails over carbon steel in most cases in terms of girder replacement. A709-50CR is more cost-effective than carbon steel in achieving a high reliability/low risk level even when the cost of A709-50CR is at the high end of the spectrum.
- Applying frequent repainting on carbon steel bridges and building A709-50CR bridges are both cost-effective life-cycle maintenance strategies. The cost-effectiveness of building A709-50CR bridges compared with frequent repainting actions hinges upon the service life of the bridge and the cost of A709-50CR steel.

COST-EFFECTIVENESS OF A709-50CR ON A BRIDGE NETWORK LEVEL

After conducting risk-based optimization to determine optimal maintenance strategy for a bridge network located in Chester County, PA, subjected to corrosion, the conclusions and recommendations on the cost-effectiness of A709-50CR at a bridge network level are drawn as follows:

- Using the deterministic user equilibrium approach to estimate life-cycle network risk can be very conservative. Therefore, adopting the maintenance strategy determined using this equilibrium may lead to a waste of financial and human resources. The traffic users' sensitivity to the travel cost may play a crucial role in the user cost estimation in the stochastic user equilibrium approach. Therefore, parametric analysis on the value of θ in the logit assignment model may be worth carrying out.
- When the target life-cycle risk is low, using A709-50CR steel can lead to a lower maintenance budget for the management of bridge network compared with using carbon steel to conduct replacement. The target risk level at which using carbon steel or A709-50CR for steel bridge replacement is equally cost-effective is contingent upon multiple factors, such as the failure consequence of the bridges in the network and the failure probabilities of each bridge, among others. The user equilibrium estimation

approach adopted may also have an impact on this specific target risk level.

• Further research work should include failure consequences evaluation in terms of user cost under other stochastic assignment models (e.g., probit assignment (Daganzo & Sheffi 1977)). Conducting on-site survey to have a better understanding of travelers' behavior when facing multiple paths may be worthwhile for large-scale networks. Failure consequences associated with injuries and fatalities as well as extra greenhouse gas emissions may be considered to conduct risk analysis in a more comprehensive manner.

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