

Cost optimization of concrete bridge infrastructure

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Abstract: Recent surveys have indicated that between 30% and 40% of all bridges in North America are in various states of deterioration. Funding is limited owing to the existence of other deficient components of the transportation infrastructure. It is clear, therefore, that the return on the available funding needs to be maximized. This paper presents a review of publications on cost optimization of concrete bridge components and systems and then continues with a review of the state-of-the-art in life-cycle cost (LCC) analysis and design of concrete bridges. The main objective of the paper is to encourage bridge engineers to move towards the increased use of advanced analysis and design optimization methods.

Key words: bridge, concrete, cost, high-performance concrete, infrastructure, life-cycle cost, optimization, prestressed girders, reliability.

Résumé : Des relevés récents indiquent qu'entre 30 % et 40 % de tous les ponts en Amérique du Nord sont à des niveaux variés de détérioration. Le financement est limité en raison de l'existence d'autres composantes déficientes de l'infrastructure de transport. Il est donc clair que le retour sur le financement disponible doit être maximisé. Cet article présente une revue des publications sur l'optimisation des coûts des composantes et des systèmes de ponts en béton. L'article poursuit ensuite avec une revue de ce qui est d'avant-garde dans l'analyse des coûts de cycle de vie et de conception des ponts en béton. L'objectif principal de cet article est d'encourager les ingénieurs en structures de ponts à utiliser d'avantage les méthodes d'analyse évoluées et d'optimisation de conception.

Mots clés : pont, béton, coût, béton haute performance, infrastructure, coût du cycle de vie, optimisation, poutres précontraintes, fiabilité.

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1. Introduction

A designer's goal is to develop an "optimal solution" for the structural design under consideration. An optimal solution normally implies the most economic structure without impairing the functional purposes the structure is supposed to serve. The trend in structural design has been towards improving the final design to the maximum degree possible. This trend in design has been made possible by a number of developments. First, the advent of relatively low cost, high-power computers and the developments in structural analysis methods and mathematical programming techniques have made both the analysis and design processes, regardless of complexity, manageable with relative ease. Second, the prac-

tical attainment of improved construction materials, such as high-performance concrete (HPC), has contributed to design progress.

Until recently, many bridge designers have kept their distance from these innovations in computer-oriented analysis and design methods and construction materials. The current status of concrete highway bridge infrastructure in North America, however, necessitates that bridge engineers must move towards the increased use of advanced analysis and design optimization methods and enhanced materials. According to the National Research Council of Canada Institute for Research in Construction (IRC 2002), about 40% of all bridges operating in Canada are older than 30 years, and most are in urgent need of replacement or rehabilitation. In the United States, the Federal Highway Administration considers that almost 30% of all bridges are structurally deficient or functionally obsolete (ASCE 2001). Funds for the rehabilitation or replacement of deteriorated bridges have not kept pace with the rate of deterioration. This makes it more critical than ever to ensure that new bridges are economical as well as safe and durable.

2. Design optimization of concrete bridges

Traditionally, the process of design in structural engineering has relied solely on the designer's experience, intuition, and ingenuity. Although this process has worked well, as evidenced by the existence of many fine structures, it has led

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to a less than rigorous approach to design optimization. It has generally led to conservative designs that were not the best possible.

Computational design optimization techniques have matured over the past four decades (Arora 1990) and can be of significant aid to the designer, not only in the creative process of finding the optimal design but also in substantially reducing the amount of effort and time required to do so. Despite this, there has always been a gap between the progress of optimization theory and its application to the practice of bridge engineering. In 1994, Cohn and Dinovitzer (1994) estimated that the published record on structural optimization since 1960 can conservatively be placed at some 150 books and 2500 papers, the vast majority of which deal with theoretical aspects of optimization. Documentation in their comprehensive catalogue of published examples shows that very little work has been done in the area of optimizing concrete highway bridges. The situation remains relatively unchanged today.

3. Previous work on optimizing concrete bridges

Lounis and Cohn (1993) defined the optimal bridge superstructure as “one of minimum total cost, using standardized girder sections and traffic loading”. In this context, they identified three levels of optimization and indicated that the overall economic impact increases with higher levels (Lounis and Cohn 1993; Cohn and Lounis 1994). These three levels are discussed in the following sections.

3.1. Level 1: Component optimization

This is the most widely reported optimization procedure because of its simplicity. It involves optimization of the cross-sectional dimensions of the components of the bridge superstructure (e.g., girders, deck slab), prestressing and nonprestressing reinforcements, and prestressing tendon profile. If standard precast girders are used, cross-sectional dimensions of the girders are known a priori and are thus eliminated from the optimization procedure.

Although minimum cost was used as a criterion for optimization in many of the earlier studies, formal optimization procedures were not utilized. The computer was simply used as a computing device to carry out the substantial amount of repetitive calculations required to find all possible combinations so that the least expensive could be chosen. This procedure does not need to be explained further, so it will not be discussed here.

Optimization techniques, on the other hand, transform the conventional design process of trial and error into a formal and systematic procedure. Optimization techniques force the designer to clearly identify the design variables that describe the design of the structure, an objective function that measures the relative merit of alternate feasible designs, and the constraints on performance that the design must satisfy. If the objective function and the constraints are linear functions of the design variables, then the problem is a linear programming problem.

In prestressed concrete design optimization, the problem is nonlinear, requiring the use of nonlinear optimization procedures. When these were in their infancy during the 1960s,

and even after they matured and became available through commercial software two decades later, it was more feasible to reduce the nonlinear programming problem into a linear one because the nonlinear software was expensive to obtain and operate and because linear programming methods were more well developed. A nonlinear problem can be linearized by representing curved boundaries in the nonlinear design space by a series of straight lines in a linear design space. Currently, design optimization software packages that can solve nonlinear programming problems are relatively inexpensive and easy to operate. It is, therefore, now more efficient to use them for prestressed concrete design optimization.

3.1.1. Linear optimization

Linear programming methods were used by Kirsch (1985, 1993, 1997) to optimize indeterminate prestressed concrete beams with prismatic cross sections through a “bounding procedure”. To simplify the problem, a two-level formulation was used to reduce the problem size and eliminate potential numerical difficulties encountered because of the fundamentally different nature of the design variables. The concrete dimensions were optimized in one level, and the tendon variables (prestressing force and layout coordinates) were determined in another level. As a first step, a lower bound on the concrete volume was established without evaluating the tendon variables. The corresponding minimum prestressing force was an upper bound. Similarly, a lower bound on the prestressing force was determined by assuming the maximum concrete dimensions. Based on the two bounding solutions, a lower bound on the objective function was evaluated. The best of the bounding solutions was first checked for optimality. If necessary, the search for the optimum was then continued in the reduced space of the concrete variables using a feasible directions technique. For any assumed concrete dimensions, a reduced linear programming problem was solved. The process was repeated until the optimum was reached.

Fereig (1985) linearized the problem of prestressed concrete design optimization to determine the adequacy of a given concrete section and the minimum necessary prestressing force. He developed preliminary design charts that could be used to determine the required prestressing force for a given pretensioned, simply supported Canadian Precast–Prestressed Concrete Institute (CPCI) bridge girder, for any given span length and girder spacing. Similar charts were later developed for some of the commonly used bridge I-girder sections in the United States (Fereig 1994a, 1996).

Jones (1985) studied the design of precast, prestressed concrete simply supported box girders used in multigirder highway bridges. The cross-sectional geometry and the grid within which strands could be placed were assumed given and fixed. The design variables included the concrete cylinder strength and the number, location, and draping of strands. The constraints imposed were stresses at prestress release and at service, ultimate moment capacity, cracking moment capacity, and release camber.

Fereig and Smith (1990) used linear programming to establish preliminary design charts for pretensioned, single-span, standard bridge box girders and hollow slabs used in Canada. The charts can be used to obtain the required

prestressing force for a given span length and spacing. The same methodology was later utilized by Fereig (1994b) to develop similar design charts for standard prestressed box girder bridges according to American Association of State Highway and Transportation Officials (AASHTO) specifications. A procedure to design the superstructure at minimum cost by minimizing the prestressing force was presented.

Recently, Fereig (1999) compared the required prestressing forces obtained in his latter study with those that would be obtained using concrete with a 69 MPa cylinder strength. It was found that using the higher concrete strength allowed a reduction in the prestressing force of from 4 to 12% depending on the girder spacing and span for the example considered.

3.1.2. Nonlinear optimization

Cohn and MacRae (1984a) studied simply supported reinforced, fully prestressed (pretensioned and post-tensioned), and partially prestressed concrete I-beams with fixed cross-sectional geometry subjected to serviceability and ultimate limit states constraints using a nonlinear programming technique. For the examples considered, they concluded that partial prestressing appears to be a more economical solution for post-tensioned members than full prestressing for a prestressing-to-reinforcing steel cost ratio of greater than 4. For pretensioned members, on the other hand, full prestressing appears to be the best solution. For partially prestressed concrete members, they concluded that the optimal solutions vary little for a prestressing-to-reinforcing steel cost ratio in the range of 0.5–6.

Cohn and MacRae (1984b) further performed parametric studies on some 240 simply supported reinforced, fully prestressed, and partially prestressed concrete beams with different dimensions, depth-to-span ratios, and live load intensities. They concluded that, in general, reinforced concrete beams are the most economical option at high depth-to-span ratios and low live load intensities. On the other hand, fully prestressed beams are the most economical option at low depth-to-span ratios and high live load intensities. For intermediate values, partial prestressing is the most economical solution.

3.1.3. Shape optimization

During the past decade, several attempts have been made toward developing new girder shapes that are structurally more efficient in continuous-span bridges, economically more competitive with structural steel, and aesthetically more pleasing than the older girder shapes. Most of these studies have resulted in girder cross sections with enlarged bottom flanges that are normally required to take full advantage of higher concrete strengths by accommodating more prestressing strands. Examples of these new improved girders are presented as follows.

In Florida, the need for continuous, long-span bridges to satisfy stringent ship impact requirements has prompted the development of a series of bulb-tee girders primarily intended for use in continuous bridges (Csagoly and Nickas 1987). In an effort to improve its appearance, this series was designed with circular curves, rather than sharp angles, at the points where the flanges join the web.

Researchers at the University of Nebraska-Lincoln have developed a series of optimized precast, prestressed concrete

bulb-tee girders, called the Nebraska University (NU) girder series (Geren and Tadros 1994). The girders were developed for optimal performance in a continuous bridge of two equal spans with full-length continuity post-tensioning. They should also perform well in pretensioned bridges with continuity achieved by mild steel reinforcement in the deck slab and in simply supported bridges. This NU girder series was also designed with circular curves, rather than sharp angles, at the points where the flanges join the web and on the outside edge of the flanges.

Aesthetics and economy were the primary design considerations for a new precast, prestressed concrete open-top trapezoidal girder with sloping webs, developed at the Texas Department of Transportation (Ralls et al. 1993). The emphasis on aesthetics came from the perception that I-shaped girders are unattractive when spaced close together with many visual break lines along the underside of the bridge. To achieve the desired aesthetics, yet maintain the economy of precast, prestressed concrete girders manufactured under controlled plant conditions, two new cross sections of the U-beam were developed such that the number of girders and the number of visual break lines are reduced by replacing the I-shaped girders with more widely spaced girders having smoother lines.

A new precast, prestressed concrete bulb-tee bridge girder series was introduced in 1997 in the New England region, which comprises six states (Bardow et al. 1997). This development was prompted by the need for a new precast girder standard for the region that would be competitive with steel and would meet the often-conflicting requirements posed by the New England environment. On the one hand, the depth, shipping length, and weight of the girders have to be kept to a minimum owing to the limitations of existing roads, particularly in rural areas. On the other hand, highway design considerations are pushing designers to use longer spans to improve safety on roadways by providing greater side clearances and also to minimize the impact on environmentally sensitive areas such as wetlands. The general features of the bulb-tee shapes developed in Florida and Nebraska were adopted for the New England bulb-tee girder series.

The most recent development in this field is the introduction of a new precast, prestressed concrete bulb-tee bridge girder series in Washington State (Seguirant 1998; Van Lund et al. 2002). The primary design consideration for this series was deeper sections that will span farther than previously available Washington Department of Transportation (WSDOT) sections. Wider spacings and fewer girder lines can also be achieved with the new girders for spans in the range of previous WSDOT standard girders. The sections are available in single-piece, pretensioned and multiple-piece, post-tensioned segmental versions.

In Europe, some attempts have been made to develop more efficient precast girder shapes. In the United Kingdom, for example, the Y-beam series was proposed in the early 1990s (Taylor et al. 1990). Each beam in this series resembles an inverted T-beam. This series is suitable for medium-span bridge construction, particularly for continuous bridges.

3.2. Level 2: Configuration or layout optimization

Configuration or layout optimization is concerned with finding the optimal combination of longitudinal and trans-

verse component arrangement within a given bridge system. Specific items considered include the number of spans, the position of intermediate supports, the restraint conditions (simply supported or continuous), the number of girders, etc. Much less work has been reported on this type of optimization than on component optimization.

Torres et al. (1966) used linear programming methods to develop a computer program for the optimal design of single-span, precast, prestressed concrete AASHTO girder bridges. The program was used to produce a series of design charts to obtain the optimal type and number of girders for a range of bridge span lengths and widths and live load specifications.

Aguilar et al. (1973) developed a computer program that could be used to evaluate a multitude of preliminary designs and select the most economical configuration for a multi-span, simply supported slab-on-girder bridge. This included the number of spans and their lengths as well as the number of girders and their types chosen from the AASHTO cross sections. The criterion used for optimization was the minimum total cost of the bridge including superstructure and substructure. The program was designed to handle constraints such as terrain geometry and soil profiles.

3.3. Level 3: System optimization

System optimization involves the optimization of the overall features of the bridge structural system, including materials, structural type and configuration as well as component sizes. This is the most complex optimal design problem and very few attempts to solve it are known.

Aparicio et al. (1996) developed a computer-aided design system for prestressed concrete highway bridges. The system designs precast, pretensioned and post-tensioned, multispan, simply supported I-girder bridges, solid or cellular slab bridges with single or multiple spans, and continuous, segmental box girder bridges with variable depth erected by the balanced cantilever method. Starting from basic geometrical data, the program provides the complete geometry, prestressing and nonprestressing steel areas, amounts of materials, and costs of all bridge components (deck, girders, bearings, piers, abutments, and foundations). Optimization routines are applied to adjust the final design. The program could be a useful decision-making tool for both design and administration engineers because it is easy to perform sensitivity analyses to assess the influence of any of the variables on the final cost of the bridge.

Lounis and Cohn (1993) optimized short- and medium-span precast, pretensioned concrete slab-on-girder bridge systems. Simply supported I-girder bridges (with one, two, and three equal spans) and continuous I-girder bridges (with two and three equal spans) were considered. Span lengths were varied between 10 and 30 m. Continuity was achieved by mild steel reinforcement in the deck slab. Some of the standard CPCI and AASHTO cross sections were used. Non-linear programming methods were utilized to obtain the minimum superstructure cost, as a criterion for optimization, for the different bridge configurations investigated. Based on the resulting optimal solutions, some standards were developed for selecting optimal I-girder bridge systems for various ranges of geometry. For example, it was found that the

optimal longitudinal configuration for bridge lengths not exceeding 27 m was a single span. For bridge lengths over 27 m, the optimal superstructure system was, in general, a two-span continuous system. Such standards could serve both as guidelines for preliminary designs and as yardsticks against which final bridge designs could be evaluated. The optimal solutions presented in this study, however, were based on a concrete cylinder strength of 40 MPa; their validity for higher strengths is questionable.

Hassanain and Loov (1999) studied the potential economic benefits from the use of HPC with concrete cylinder strengths of up to 100 MPa for continuous, precast, pretensioned CPCI I-girder highway bridges. The problem was formulated as an optimal design problem (Arora 1989). Design variables included the required prestressing force, tendon eccentricities, amounts of nonprestressed flexural reinforcement, girder concrete cylinder strength, and deck slab thickness. The objective function was based on the cost per unit area of deck:

$$[1] \quad \text{Cost} = \{n_g C_g + C_c V_c + C_s [m_s + n_p (m_{sn} + m_{sp}) + m_p]\} / WL$$

where C_c is the cost of concrete in the deck per unit volume; C_g is the cost of one girder, including materials, production, transportation, and erection; C_s is the cost of nonprestressing steel per unit mass; L is the total length of the bridge; m_p is the mass of positive moment steel in the girders; m_s is the mass of nonprestressing steel in the deck; m_{sn} is the mass of negative moment steel in the deck at each pier; m_{sp} is the mass of positive moment connection steel at each pier; n_g is the number of girders; n_p is the number of positive moment connections at piers; V_c is the volume of concrete in the deck; and W is the width of the bridge deck. Constraints included functional constraints (serviceability and flexural ultimate limit state provisions of the Ontario Highway Bridge Design Code (OHBDC 1992)) and practical constraints (limits for design variables) (Hassanain 1998). The design optimization problem was solved using a nonlinear programming method. Some 115 optimal superstructure designs were generated for a two-equal-span, continuous, slab-on-girder bridge that is representative of existing bridges of this type. The bridge had three traffic lanes with an overall width of 12 m (Hassanain and Loov 1999).

The study recommended several strategies for reducing slab-on-girder bridge costs by using HPC. It was concluded that if four girder lines are needed, concrete cylinder strengths ranging from 40 to 80 MPa will be optimal, depending on the span length. A strength greater than 100 MPa is not beneficial unless the span is longer than 52 m or shallower girders are desired. If three girder lines are allowed in a bridge of this width, strengths up to 100 MPa will be optimal, and a lower superstructure cost will be achieved compared with the same span with four girder lines. For spans up to 42 m, two girder lines will result in the lowest superstructure cost, with the optimal concrete cylinder strength varying from 40 to 100 MPa.

4. Life-cycle cost analysis

Life-cycle cost (LCC) analysis is a process for evaluating the total economic worth of a project by analysing initial costs plus discounted future costs, such as maintenance, rehabilitation, and reconstruction costs over the lifespan of the project. An important expansion of the LCC concept has been recently made by taking into account user costs in addition to costs accruing to bridge owners. User costs represent the societal costs that are imposed when the serviceability of a bridge is reduced during routine maintenance operations or eliminated during major repairs. Examples of user costs include those associated with travel delay, vehicle operation, and accidents. Studies have shown that user costs make up a very sizeable component of total LCC estimates (Chang and Shinozuka 1996).

Life-cycle cost analysis is based on the concept of discounted cash flow. The costs incurred and the benefits gained by the owner and users throughout the entire life of a bridge are estimated and converted into a single equivalent cost (such as the present worth, annual worth, or future worth) for the purpose of comparison of various alternatives (Sobanjo 1999). Bridge management systems typically employ the present worth, defined by eq. [2], as the equivalent cost, where PW is the present worth, F is the future cost, r is the discount rate, and t is the timing of the future expenditure from the time of construction (in years).

$$[2] \quad PW = F(1 + r)^{-t}$$

Existing regulatory requirements call for consideration of LCC concepts for civil infrastructure investments in the United States. For example, the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA 1991) and the National Highway System Designation Act of 1995 (NHSDA 1995) mandate that LCCs, rather than initial costs alone, be considered in the design and engineering of bridges, tunnels, and pavements in state- and metropolitan-level transportation planning. In Canada, the City of Winnipeg, Manitoba, recommends that LCC analysis be used for all decisions related to infrastructure alternatives and that maintenance be deferred only if the impact on life expectancy and LCCs is minimized and if maintenance is factored into initial infrastructure costs (Vanier 2001). The integration of these requirements in the design of bridges has been very limited until now (Ehlen 1997).

4.1. Significance of life-cycle cost analysis and design

In 1992, Veshosky and Beidleman (1992) published a one-page article in which they expressed concerns regarding the practicality of applying LCC analysis to bridges. They cited the lack of reliable, consistent cost data collected on a systematic basis for a variety of bridges, the changes in design criteria and techniques, the variability in construction materials and methods, and other factors contributing to costs. Veshosky and Beidleman argued that the use of LCC criteria in the evaluation of bridge construction alternatives could introduce as much uncertainty as it is intended to solve!

Despite these concerns, LCC analysis can still provide a significant and powerful approach if it is regarded not as an "exact science", but rather "as a method that allows mainte-

nance costs to be studied in parametric fashion so that they can be given a truer weighting in the design process and not forgotten entirely" (Leeming 1993). Conceptually, the approach can be used in the process of design selection for new bridges and in the optimal operation and maintenance of existing ones. More importantly, the exercise of LCC estimation provides insight into the various cost components and identifies the specific information required to make such an estimation (Chang and Shinozuka 1996).

To achieve a sustainable level of service for bridges in the 21st century, it is apparent that funding for bridge maintenance and repair will have to be increased significantly. Recognizing the funding constraints that exist owing to the other deficient components of the infrastructure, it is clear that the return on the available funding needs to be maximized. Use of LCC analysis provides a means of structure selection on the basis of the least long-term cost to society (Freyermuth 2001).

4.2. Obstacles to applying life-cycle cost concepts to bridges

The two primary obstacles to applying LCC concepts to highway bridges have been the lack of access to reliable cost and service-life data and the lack of a commonly accepted methodology (Hawk 1998; Arditì and Messiha 1999). In what follows, an overview of some of the efforts to remove these obstacles is presented.

4.2.1. Cost data

To minimize LCCs, it is necessary to quantify the cost associated with the several stages of life of the structure. The design problem remains an optimization problem, but the discounted costs of inspection, maintenance, and repair have to be added to those previously considered. Several cost functions exist in the literature (e.g., Chang and Shinozuka 1996; de Brito and Branco 1997, 1998; Frangopol et al. 1999; Koskisto and Ellingwood 1997; Thoft-Christensen 2000). In this paper, the global cost function developed by de Brito and Branco (1997, 1998) is presented as an example.

The cost function, C , includes the structural costs, C_{ST} , and the functional costs and benefits, C_{FU} , during the structure's life cycle:

$$[3] \quad C = C_{ST} + C_{FU}$$

The structural costs include the initial costs of design and construction, C_0 , inspection costs, C_I , current maintenance costs, C_M , repair costs, C_R , and structural failure costs, C_{FSF} :

$$[4] \quad C_{ST} = C_0 + C_I + C_M + C_R + C_{FSF}$$

For new bridges, most of the structural costs can be approximately predicted based on current cost indices, historical data from similar bridges, and (or) expert opinions. Sobanjo (1999) argued that historical data could have inherent statistical randomness and that expert opinions could introduce some subjectivity into the final estimates. He suggested that statistical data randomness can be adequately handled in a decision analysis through the use of probability theory and that subjectivity in estimates can be accounted for using the concepts of fuzzy sets theory. He presented a

set of conceptual algorithms and equations to illustrate how those uncertainties can be handled in bridge LCC analysis.

An item in eq. [4] that requires special consideration is the structural failure costs, C_{FSF} . These include all costs resulting from a structural collapse of the bridge. Although collapse does not occur under normal circumstances, these costs can still be considered in an economic analysis analogous to insurance costs. The structural failure costs can be obtained from the probability of failure, P_f , and the cost of the actual collapse, C_{FF} , as follows:

$$[5] \quad C_{FSF} = P_f C_{FF}$$

The functional costs and benefits can be expressed as follows:

$$[6] \quad C_{FU} = C_{FFF} - B$$

where functional failure costs, C_{FFF} (referred to earlier in this paper by the more common term "user costs"), are those associated with reductions in the service conditions of a bridge, such as speed limitation and load reduction, and the benefits, B , correspond to negative functional failure costs because they are associated with an improvement of the service level in the structure.

The functional failure costs include the following components: (i) the costs due to delayed traffic, caused by the slowing of the traffic crossing the bridge, especially during rush hours; (ii) the costs due to traffic being detoured from one bridge to others nearby because of the saturation of the traffic flow; and (iii) the costs of detouring heavy loads from one bridge to others because of insufficient structural capacity.

Each of these components can be divided into costs of time wasted by drivers, fuel costs, vehicle maintenance costs, and traffic accident costs. These costs have to be computed using data such as daily and yearly traffic surveys, service design level of the road, future traffic estimates, existing alternatives to each bridge, the traffic and structural capacity of the bridge, and energy and vehicle maintenance costs (de Brito and Branco 1997).

4.2.2. Service-life data

An important step in an LCC analysis is to determine the desired analysis period (i.e., the design service life or, as it is sometimes called, the planning time horizon). This period should be sufficiently long to reflect long-term differences associated with feasible design alternatives. For HPC bridges, a minimum of 100 years should be considered. In addition to determining the desired service life, the terminal serviceability level needs to be defined. Terminal serviceability (also referred to as end of functional service life) is the maximum level of degradation that will be permitted prior to replacement of the structure. To a large degree, terminal serviceability will depend on the functional requirements (e.g., structural capacity, aesthetics, smooth ride) of the bridge (Neff 1999).

The main structural components need to remain functional for the full service life with only minor repair costs. The estimation of the service life based on physical deterioration is a complex problem. It includes the definition of the reference limit states associated with the end of the service life, the environment characterization, the study of the degrada-

tion phenomena of the materials and structural components, and the definition of mathematical models for evaluation of the degradation path. In concrete bridges, the most important degradation mechanism is corrosion of reinforcement (de Brito and Branco 1997).

Currently, there are no specific code recommendations in terms of easily measurable concrete properties (e.g., minimum cement content, maximum water to cementitious materials ratio, minimum strength) and reinforcement cover that would guide designers to achieving the expected durability for structures with service lives of 100 and more years. In such cases, the study of the service life must be based on physical deterioration models (e.g., Hjelmstad et al. 1998; Neff 1999; Stewart and Rosowsky 1998; Troive 1996; Troive and Sundquist 1998; Val et al. 2000), based on environmental conditions, and the limit states adopted for design. With these models, an estimation of the degradation process can be obtained for specific concrete properties and reinforcement cover. Normally, several alternatives may be used in the choice of concrete properties and cover, and a cost-efficiency analysis performed to reach a decision (de Brito and Branco 1997).

For important bridges, a monitoring system should be installed. Monitoring during construction and service combined with periodic inspections will allow the confirmation or adjustment of the deterioration rates assumed in the design. Currently, condition assessment of bridges is an ongoing task for transportation authorities. Several bridge management systems, such as OBMS in Canada (Thompson et al. 1999) and PONTIS and BRIDGIT in the United States (Khan 2000), have been developed during the past decade to assist in the task of acquiring and interpreting inspection data from North America's bridges. The sharing of information among jurisdictions should lead to a large database of service-life data.

4.2.3. Analysis methodologies

Several methods for the implementation of LCC analysis have been presented in the literature. A brief review of some of the recent methods is presented.

Mohammadi et al. (1995) combined the present worth, PW, with a new parameter they called the value index, VI, which can be used to help quantify the bridge decision-making process. Specifically, VI describes three major elements of a bridge LCC analysis: (i) bridge or bridge-element condition rating, (ii) the cost associated with various bridge expenditures, and (iii) bridge service-life expectancy. A condition rating range from 1 to 9 (with 1 representing the worst condition) can be used. Ideally, VI can be formulated in terms of the three independent parameters described previously. The VI can then be used as the objective function in an iterative optimization scheme in which various constraints on the parameters can be imposed. Using this approach, the option with the greatest VI and the least PW is taken to be the most desirable one. The advantage of using both VI and PW instead of PW alone during LCC analysis is that VI also includes the optimum time schedule for the selected bridge expenditures and the total cost in the analysis.

Recently, there has been more interest in including bridge lifetime reliability in the process of optimizing investments based on life-cycle costing (e.g., Ang et al. 2001; Estes and

Frangopol 2001; Frangopol 1999; Frangopol and Estes 1997; Frangopol et al. 1998; Jiang et al. 2000; Koskisto and Ellingwood 1997). It has been reported that deterministic condition assessment of bridges can be overconservative, leading to an overestimation of the need for bridge repair (Frangopol et al. 1999). Also, because of the uncertainties related to material properties, structural dimensions and models, live loads, and environmental conditions, in addition to the uncertainties related to the deterioration of concrete bridges, an assessment based on probabilistic (i.e., reliability) modelling of the significant parameters seems to be warranted (Val et al. 2000).

Frangopol et al. (1999) presented an LCC analysis methodology that integrates maintenance, repair, and replacement decisions in bridge management based on reliability, optimization, and life-cycle costing. Using this methodology, maintenance, repair, and replacement actions can be based on an acceptable level of risk quantified using structural reliability methods. Optimum maintenance, repair, and replacement strategies can be identified that provide safety and serviceability at minimum expected LCC. As an example, Frangopol et al. indicated that the dimensions of bridge elements could be modified to increase the time between repairs and to reduce the total number of repairs over the life of the bridge. The dimensions of the bridge elements are identified using optimization techniques, and a reliability-based optimum maintenance strategy is identified that minimizes the total LCC. It was mentioned briefly that this approach can be used to compare several feasible bridge designs for a variety of materials.

Neff (1999) presented a methodology for incorporating reliability concepts in predicting the LCC of concrete bridges subject to corrosion of the reinforcement. The methodology is applicable to all corrosion protection systems. An example of LCC analysis for selecting a corrosion protection system for a reinforced concrete bridge deck was presented. Five different protection strategies were evaluated and compared against a base case of no corrosion protection. These strategies included epoxy coating one or both mats of reinforcement, stainless steel rebar, galvanized steel rebar, and a combination of HPC and epoxy-coated rebar. The alternatives were compared using an analysis period of 75 years. It was found that the combination of HPC and epoxy-coated rebar was the most economical corrosion protection alternative over the life of the bridge.

5. Concluding remarks

The lack of funds available to repair and replace deficient bridges necessitates that the return on future transportation investments has to be maximized through decision-making processes that take into consideration the expected maintenance, repair, and replacement costs and not just the lowest initial cost. It is more critical than ever to ensure that new bridges are economical as well as safe and durable. New construction materials such as HPC together with improved beam shapes could play significant roles in the renewal of the deteriorated bridge infrastructure in North America.

Computational design optimization techniques have matured over the past four decades and can be of significant aid to the designer in the creative process of finding the optimal

design. Combining optimization techniques and LCC analysis can facilitate better decision-making, thereby managing risk and optimizing investments. Much work has been done during the past decade to overcome the obstacles to applying LCC to highway bridges. However, very few researchers have tried to quantify the long-term benefits from the use of HPC for bridges based on life-cycle costing. There is clearly a need for more research in this area.

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List of symbols

- B benefits
- C global cost function
- C_0 initial costs of design and construction
- C_c cost of concrete in deck slab per unit volume
- C_{FF} cost of failure
- C_{FFF} functional failure costs
- C_{FSF} structural failure costs
- C_{FU} functional costs and benefits
- C_g cost of one girder, including cost of materials, production, transportation, and erection
- C_I inspection costs
- C_M current maintenance costs
- C_R repair costs
- C_s cost of nonprestressing steel per unit mass
- C_{ST} structural costs
- Cost minimum superstructure cost per deck area
- F future cost
- L total length of a bridge
- m_p mass of positive moment steel in girders
- m_s mass of nonprestressing steel in a deck slab
- m_{sn} mass of negative moment steel in a deck slab at each pier
- m_{sp} mass of positive moment connection steel in a deck slab at each pier
- n_g number of girders
- n_p number of positive moment connections at the piers
- P_f probability of failure
- PW present worth equivalent cost
- r discount rate
- t timing of future expenditure from time of construction (in years)
- V_c volume of concrete in a deck slab
- VI value index
- W width of a bridge