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Life-Cycle Cost Analysis of Alternative Reinforcement Materials for Bridge Superstructures Considering Cost and Maintenance Uncertainties

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Abstract

A life cycle cost analysis (LCCA) was conducted on prestressed concrete bridges using carbon fiber reinforced polymer (CFRP) bars and strands. Traditional reinforcement materials of uncoated steel with cathodic protection and epoxy-coated steel were also considered for comparison. A series of deterministic LCCAs were first conducted to identify a range of expected cost outcomes for different bridge spans and traffic volumes. Then, a probabilistic LCCA was conducted on selected structures that included activity timing and cost random variables. It was found that although more expensive initially, the use of CFRP reinforcement has the potential to achieve significant reductions in life cycle cost, having a 95% probability to be the least expensive alternative beginning at year 23-77 after initial construction, depending on the bridge case considered. In terms of life cycle cost, the most effective use of CFRP reinforcement was found to be for an AASHTO beam bridge in a high traffic volume area.

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Introduction

In the United States, it has been estimated that approximately 30% of the nation's bridges require immediate repair due to the effects of corroding reinforcement, at an estimated cost of over $8 billion (Won et al. 2007; FHWA 2001). The main cause of this corrosion is exposure to chlorides, which are often present in deicing chemicals as well as seawater. Corrosion produces products that increase the volume of the steel by 3-6 times, damaging the surrounding concrete as the corroding steel expands. This has long been recognized as a significant and costly maintenance problem for concrete bridge components, and various methods of damage mitigation have been attempted. Some of these include the use of admixtures and changing the concrete mix design to prevent chloride penetration or action; increasing concrete cover over reinforcement; cathodic protection; and the use of epoxy-coated reinforcement, among others. In general, these methods have been met with limited success (FHWA 2001; Smith and Virmani 1996).

In light of this problem, in the last two decades interest in non-corrosive alternatives such as fiber reinforced polymer (FRP) composites have grown, replacing traditional steel reinforcement in a small number of bridges. Although not codified in the American Association of State and Highway Transportation Officials (AASHTO) Bridge Design Specifications (2007) or Building Code Requirements for Structural Concrete, ACI-318 (ACI 2008), publications developed by the American Concrete Institute, the Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars, ACI-440.1R (ACI 2006), and Prestressing Concrete Structures with FRP Tendons, ACI 440.4R (ACI 2004), provide design guidance for use of FRP in place of steel reinforcement. A similar document, specifically for use of glass FRP bars in bridge decks, was recently produced by AASHTO (2009).

Presently, various examples of FRP reinforcement exist in bridges around the world. Some of these in the US (followed by year of construction) are on bridges located at Pierce Street in Lima, OH (1999); Salem Avenue in Dayton, OH (1999); Rollins Road in Rollinsford, NH (2000); Sierrita de la Cruz Creek in Potter County, TX (2000); 53rd Avenue in Bettendorf, IA (2001); Bridge Street in Southfield, MI (2001); Highway 151 in Waupun, WI (2005); Route Y in Boone County, MO (2007), as well as others.

Because the initial construction cost of a bridge reinforced with FRP is often significantly higher than when using steel reinforcement, any potential economic advantages associated with FRP will not be realized unless costs over an extended period of time are considered. That is, the potential reduction in maintenance costs associated with using non-corrosive FRP may eventually outweigh the higher initial cost of construction, as compared to a steel-reinforced structure. Thus, a life-cycle cost analysis (LCCA) should be performed to determine if and when an eventual cost-savings occurs. Using this approach assists transportation agencies to quantify the economic impact, as a function of time, of bridge reinforcement alternatives. As construction costs and the timing and associated costs of many maintenance events are rarely known with certainty, an important component of LCCA is consideration of these uncertainties. Inclusion of uncertainties, by representing critical LCCA parameters as random variables, allows for results to be expressed in a probabilistic sense; for example, the probability that one reinforcement alternative is less costly than another as a function of time.

A large body of work on the LCCA of various civil engineering structures and facilities has been conducted in the last two decades, and numerous studies have applied LCCA to bridge structures. Much work focused on bridges involved the evaluation of cost effectiveness of component replacement options (Fagen and Phares 2000) or treatment methods for specific deteriorating bridge components, which may involve corrosion of reinforcement steel in concrete structures (Mohammadi et al. 1995; Bhaskaran et al. 2006) or steel girder bridges (Zayed et al. 2002; Weyers and Goodwin 1999). Various authors utilized LCCA to develop bridge management tools, such that lowest life time costs of decks and other components could be obtained by optimizing maintenance activities (Rafig et al. 2005; Hegazy et al. 2004; Kaito et al. 2001; Huang et al. 2004), while NCHRP Report 483 (TRB 2003) outlined the general methodology for bridge structure LCCA. More recently, researchers have attempted to include environmental costs in LCCA to consider bridge structure sustainability (Geryasio and da Silva 2008; Kendall et al. 2008). LCCA has also been performed on pre-cast composite bridge decks (Hastak et al. 2003; Ehlen and Marshall 1996; Ehlen 1999; Meiarashi et al. 2002; Nystrom et al. 2003; Chandler 2004). Additional work on LCCA was conducted that emphasized inclusion of cost, deterioration, and load uncertainties (Frangopol et al. 2001; Thoft-Christensen 2009; Daigle and Lounis 2006; Furuta et al. 2006). However, other than initial work done by the authors (Jensen et al. 2009), there has been no available life-cycle cost analysis for the use of CFRP reinforcement bars and strands in prestressed concrete bridges in place of steel, particularly when considering scheduling and cost uncertainties. The present investigation is significant as it considers the selective use of CFRP in place of steel reinforcement in concrete bridge structures, whereas earlier studies have focused on replacing entire bridge components (i.e. steel and concrete) with composite materials. The approach considered here represents an outcome of significantly greater economic feasibility than that shown from previous investigations (Nystrom et al. 2003; Ehlen 1999).

Therefore, the goal of this study is to determine whether prestressed concrete bridges utilizing CFRP reinforcing bars and strands can represent a cost effective design alternative to conventional steel reinforced prestressed concrete bridges. The specific objectives of this study are to: 1) determine the life cycle cost (LCC) of typical prestressed concrete bridges reinforced with uncoated steel, epoxy-coated steel, and CFRP, considering cost and maintenance uncertainties, and; 2) determine the probability that the CFRP-reinforced alternative is a less-costly alternative as a function of time.

Structures Considered

It was desired that a range of practical bridge configurations be represented that might be favorable, unfavorable, and typical, for the LCC of the reinforcement alternatives considered. Therefore, before the more computational costly probabilistic LCCA was conducted, a series of deterministic LCCAs were completed on various bridge and traffic configurations to identify cases that would provide a reasonably wide range of outcome possibilities. Combinations of two bridge girder types, three span lengths, and two or three traffic volumes, depending on span, were considered, for a total of 26 deterministic LCCAs.

The girder types considered were side-by-side prestressed concrete box beams and prestressed AASHTO beams. The box beam bridge was based on an existing typical two-lane design used by Michigan DOT (MDOT). This is a precast, prestressed bridge with transverse post-tensioning, for which the original construction drawings were available. The bridge is located in Oakland County in South East Michigan, and carries South Hill Road over Interstate Highway 96. At this location, South Hill Road (on bridge) has two lanes with shoulders while I-96 (below bridge) has three lanes both directions. The bridge is composed of two 122 ft long simple spans for a total length of 244 ft. The deck slab is 45 ft. wide and 6 in. thick, with a single layer of reinforcement. The bridge is composed of eleven side-by-side precast prestressed box beams, each with cross-sectional area as shown in Figure 1. In addition to this 122 ft. span bridge, short span (45 ft) and medium span (60 ft) versions of this structure were also considered for analysis. For these two other cases, the structural members of the original long span bridge were redesigned for these new lengths. The existing bridge as well as the two shorter-span hypothetical structures were designed according to the Michigan Bridge Design Manual (2001, 2003), which is based on the AASHTO LRFD Bridge Design Specifications (1998). Similarly, hypothetical long-span, medium-span, and short-span prestressed concrete AASHTO beam bridges were designed based on MDOT practices which have the same overall geometries as the box beam bridges except that the slabs are 9 in. thick. The medium-span AASHTO beam bridge has a cross-section which is shown in Figure 2.

Two cases of traffic volume on each bridge were considered: low volume, with an initial annual average daily traffic (AADT) of 1,000, and a high volume, with initial AADT of 10,000. The annual growth rate was taken as 2% and limited to a maximum AADT of 26,000, which was calculated from the free flow lane capacity of the roadways using the Highway Capacity Manual (TRB 2000). Below bridge traffic volumes are given in Table 1, with the annual growth rate taken as 1%. The short, medium, and long span bridges were assumed to span over 4, 6, and 8 lanes of traffic, respectively.

These combinations of bridge girder type, span, and traffic volumes resulted in the 26 cases for deterministic LCCA. For each of these cases, three reinforcing alternatives were considered, which is the focus of this study: (a) black (i.e. without epoxy-coating) steel reinforcement with cathodic protection; (b) epoxy-coated steel reinforcement; and (c) CFRP reinforcement. The CFRP bridge is designed based on ACI 440.1 (2006) and ACI 440.4 (2004) design guidelines and uses typical CFRP reinforcing bar properties (with strength of 140-150 ksi). The CFRP is designed such that it has the same flexural and shear design capacities as the steel reinforced bridges.

Based on a deterministic version of the activity timing schedule and costs detailed in the sections below (i.e. treating maintenance time and cost random variables as deterministic values equal to their means), it was found that the black and epoxy-coated steel reinforcement cases resulted in little differences in LCC from each other, but were significantly different from the CFRP case. Traffic volume was the most influential parameter, as traffic delays due to maintenance may result in significant user costs. The case found least favorable to CFRP was a low traffic volume below and on the bridge ("LL" case); the case most favorable to CFRP was high traffic below and on the bridge ("HH" case); and a typical result for CFRP was that for medium traffic below and low traffic on the bridge ("ML" case). The medium span bridges represented the range of these cases. Therefore, for the probabilistic LCCA, the medium span bridges (of both girder types) were chosen for consideration with traffic volumes of LL, ML, and HH, for a total of six cases.

Life Cycle Cost Model

The LCCA includes costs and activity timing for initial construction, inspection, repair and maintenance, demolition, replacement, and the associated user costs.

Activity Timing

As suggested by FHWA (2002), the analysis period must be long enough to include major rehabilitation actions for each reinforcement alternative. To satisfy this requirement, the LCCA was conducted up to 100 years. However, the results are presented cumulatively for each year until year 100, so the LCC for any lesser period of time can be referenced.

For consistent LCC comparison among cases, it is important that the maintenance actions are scheduled such that the expected bridge condition, at any year, is the same for all three reinforcement alternatives. In order to maintain the same performance level, different operation, maintenance and repair (OM&R) strategies may be defined for each type of bridge reinforcement alternative considered.

Bridge deterioration is driven by material deterioration, fatigue and overloading. In steel reinforced concrete bridges, the major damage that the use of CFRP attempts to mitigate is corrosion-induced. Models for corrosion-based concrete deterioration have been developed (for example, see Vu and Stewart 2005; Val 2007). However, although the available deterioration models are useful, they cannot account for the multitude of factors that affect a DOT's response to the deterioration, and therefore may not predict actual maintenance activity timing well. Therefore, for the black steel (with cathodic protection) and epoxy-coated steel bridges, the OM&R strategies in this study are based on MDOT practices for the time intervals for inspection, deck and beam-related maintenance work, and superstructure demolition and replacement. Currently, MDOT makes no maintenance scheduling distinction between bridges using cathodic-protected black steel and epoxy-coated reinforcement. For the AASHTO beam bridges, the activity timing schedule is identical to that of the box beam bridges, except that the deck replacement work is replaced by a deck deep overlay. According to MDOT, a steel-reinforced highway bridge has an expected superstructure service life of about 65 years, with various anticipated maintenance activities throughout this service lifetime.

The random variables (RVs) representing maintenance activity timing that are used in the LCCA for steel-reinforced bridges are given in Table 2. Note that, the scheduling RVs are not independent, as the scheduling of one activity depends on the time of completion of another. This is summarized in the "Initialized From" column in Table 2. RVs are normally distributed. Mean values for activity timing RVs were based on current MDOT maintenance scheduling practices, while coefficients of variation (COV) were calculated from a sample of 32 prestressed concrete highway bridges in the MDOT inventory for which historic scheduling information was available. The structures were similar in age (all built in the 1960's), geographic location (SE Michigan), as well as traffic volume (all on major interstate highways) and structural configuration (AASHTO beam) to the structures considered in this study. The mean values of the maintenance activity timing RVs over the 100 year LCCA period are shown graphically in

Figure 3(a). As MDOT has no CFRP reinforced bridges in their inventory, the OM&R strategies of existing CFRP bridges in Japan (ACC 2002; Itaru et al. 2006) and Canada (Fam et al. 1997) were consulted to establish an expected maintenance schedule for mean timing activities. Based on these schedules, the CFRP bridge is only expected to require one deck shallow overlay and one deck replacement during its service life, as shown in Figure 3(b). The mean values of these RVs are taken as 50 and 80 years, respectively, with COVs taken from Table 2 for the corresponding steel reinforced bridge case. This greatly-reduced maintenance activity is expected, as the purpose of using CFRP is the elimination of corrosion-induced concrete component deterioration. Based on MDOT practices, inspection scheduling in general does not vary, and is taken as a deterministic activity that occurs every other year for routine inspection and every 5 years for detailed inspection for the steel reinforced bridges, with a detailed inspection every 10 years for the CFRP bridge (but not during years of superstructure replacement).

Agency Costs

Agency (i.e. DOT) costs include material, personnel, and equipment costs associated with initial construction, routine and detailed inspections, cathodic protection for black steel, deck patch, deck overlay, deck replacement, beam end repair, beam replacement, superstructure demolition, and superstructure replacement.

Agency cost random variables are given in Table 3 for the black steel (BS), epoxy-coated steel (EC), and CFRP reinforced cases, which are taken as normally distributed. Many of these mean variable costs are based on a combination of sub-costs. Mean material costs such as concrete, steel reinforcement, and CFRP are based on 2009 estimates from MDOT and CFRP producers. Remaining mean costs are based on MDOT estimations as well as other sources (ACC 2002; MDOT 2006, 2008). To compute the COVs associated with agency costs, a pool of data was gathered from various relevant sources. Construction cost COVs were based on an analysis of bridge and building project cost variances (Saito et al. 1988; Skitmore and Ng 2002), where repair and maintenance cost COVs were taken from DOT bridge repair cost records (Sobanjo and Thompson 2001).

User Costs

During construction and maintenance work, traffic delays as well as increased accident rates occur. The resulting delay costs caused by construction work include the value of time lost due to increased travel time as well as the cost of additional vehicle operation. Therefore, mean user cost is taken as the sum of travel time costs, vehicle operating costs, and crash costs. Equations (1) - (3) are used to calculate these costs (Ehlen 1999).

$$\text{Travel time costs} = \left(\frac{L}{S_a} - \frac{L}{S_n}\right) \times AADT \times N \times w \tag{1}$$

$$\text{Vehicle operating costs} = \left(\frac{L}{S_a} - \frac{L}{S_n}\right) \times AADT \times N \times r \tag{2}$$

$$\text{Crash costs} = L \times AADT \times N \times \left(A_a - A_n\right) \times c_a \tag{3}$$

where L = length of affected roadway over which cars drive; S_a = traffic speed during road work; S_n = normal traffic speed; N = number of days of road work; w = hourly time value of drivers; r = hourly vehicle operating cost; c_a = cost per accident; A_a and A_n = accident rate during construction and normal accident rate per million vehicle-miles, respectively.

The values for these parameters are given in Table 4. The annual average daily traffic (AADT) value for each year of the analysis period is computed from the initial AADT and the traffic growth rate (given earlier), as limited by maximum AADT. Other parameter values are taken from the available literature (Ehlen and Marshall 1996; Ehlen 1999; Huang et al. 2004; MDOT 2010; AAA 2008; USDOT 2002). Travel time cost COV (0.12) was based on an analysis of USDOT-compiled data (USDOT 1997), while vehicle operating cost COV (0.18) was computed from average operating costs of different types of vehicles (AAA 2008; USDOT FHWA 2007). COV of vehicle crash costs (0.13) was taken from FHWA-compiled data of crash geometries pertinent to bridge work sites (FHWA 2005). The resulting user cost RVs are given in Table 5, and are taken as normally distributed. The scope of this paper excludes user costs associated with environmental damage and effects on local businesses.

Life Cycle Cost

The total life cycle cost is the sum of all yearly partial costs. Because dollars spent at different times have different present values (PV), future costs at time t, C_t, are converted to consistent present dollar values by adjusting future costs using the real discount rate r, and then summing the results over T years:

$$\mathrm{LCC} = \sum_{t=0}^{T} \frac{C_t}{(1+r)^t} \tag{4}$$

The real discount rate reflects the opportunity value of time and is used to calculate the effects of both inflation and discounting. The real discount rate is taken as 3% (FHWA 2002). For this study, the initial construction cost occurs in year 0, while the first year after bridge construction is defined as year 1. The costs associated with any subsequent activity are presented in terms of present value considering the real discount rate.

LCCA Process

For each bridge configuration considered for probabilistic analysis (box and AASHTO beam medium-span bridges with HH, ML, and LL traffic volumes), Monte Carlo Simulation (MCS) was used to first generate a simulated activity timing. Then, simulated costs are generated. For each bridge case and reinforcement option considered, 100,000 simulations per year were conducted. The specific LCCA approach was conducted as follows, for each MCS simulation *i*:

1. A maintenance schedule for the bridge is generated based on sampling the timing RVs with statistical parameters and relationships given in Table 2. This schedule will look similar to those presented in Figure 3, but with specific times for simulation *i* as determined by the random samples.

2. Once the maintenance schedule is generated in step 1, for each year *j*, MCS is used to simulate RV costs that occur in year *j*, using the RV statistical parameters described in Tables 3 and 5, as needed for that year. For years greater than 0, the cumulative cost at year *j* is determined by converting previous yearly costs to present value and summing the results up to year *j* using eq. 4. Cumulative costs for all years $j = 0$ to 100 are determined in this manner.

3. Steps 1-2 are repeated three times, once for each of the three reinforcement alternatives (BS, EC, CFRP) considered for comparison.

4. To conduct the probabilistic analysis, a limit state function (g) is needed. In this study, the limit state function of interest is in terms of cost. There are various equivalent ways this can be written, with the most direct as: $g_j = C_{CFRP} - C_{alternative}$, where C_{CFRP} is the cumulative cost of the CFRP-reinforced bridge, and $C_{alternative}$ is the cumulative cost of the bridge with black or epoxy-coated steel reinforcement, whichever is being considered for comparison, in year j. If $g_j < 0$, then C_{CFRP} was found to be cheaper for that year considered for simulation i. This result (i.e. if $g_j > 0$ or $g_j < 0$) is recorded for each year j.

5. Steps 1-4 are repeated for i=1 to 100,000 simulations. The cost probabilities (P) for each year j can then be determined with the traditional MCS process using eq. 5.

$$P\left(C_{CFRP} < C_{alternative}\right)_j = \frac{(\#\,of\ times\ g < 0)_j}{(total\ simulations;100{,}000)} \tag{5}$$

Results

Table 6 provides a summary of the mean initial (at year 0) and life cycle costs (at year 100) of the reinforcement alternatives. As shown, the use of black steel (BS) or epoxy-coated steel (EC) generally does not result in large differences in initial nor life cycle costs, with BS slightly more expensive throughout the bridge lifetime (even initially, when the cost of the first cathodic protection is included in initial bridge construction cost), with differences increasing with time to a range of about 5-11% at year 100. Relatively large differences occur when compared to CFRP, however. Here, CFRP is more expensive initially (in the worst case, up to 60% more expensive than the steel alternatives), but significantly cheaper at year 100, with the steel alternatives exceeding the CFRP bridge LCC from 53-205%. In this range, the best case for CFRP is an AASHTO beam bridge with high traffic volume, while the worst case for CFRP is on a box beam bridge with low traffic volume. Table 7 presents a detailed breakdown of the LCC at year 100. For the steel bridges, the most costly items are deck overlays, and deck and superstructure replacements. Note that for the steel reinforced bridges, user costs greatly exceed agency costs in the bridge lifetime. This is the primary reason why the reduced maintenance schedule of the CFRP bridge results in lower LCC than the steel alternatives at higher traffic volumes.

Figures 4 and 5 provide yearly cumulative life cycle cost probability results. In all cases, EC is slightly less costly than BS across the bridge lifetime. CFRP initially has a low probability of being the least costly option (from about 4–40% for the various cases), but eventually becomes the least costly option with high probability in all cases. The cumulative probability graphs confirm that use on a box beam bridge with low traffic volume is the worst case for CFRP (Figure 4), although even here it eventually becomes cheapest as well. Table 8 provides the year when the probability that the CFRP reinforced bridge cost less than the black steel or epoxy-coated alternatives is ≥ 0.5 (the expected “break-even” year), and the year when the probability that CFRP cost least is ≥ 0.95. As shown in the Table, the break-even year ranges from 6-40, with the best case for CFRP occurring on an AASHTO Beam bridge with higher traffic volume, while the worst case for CFRP occurs on a box beam bridge with low traffic volume. A similar trend appears when considering the year when CFRP is highly likely to be the least expensive alternative (i.e. ≥ 0.95), which ranges from 23-77 years after initial construction.

Conclusions

A LCCA of prestressed concrete bridges considering box beams and AASHTO beams with three levels of traffic volume was conducted. The purpose of the LCCA was to determine the cumulative life cycle costs and relative cost effectiveness of unprotected steel with cathodic protection, epoxy-coated steel, and CFRP reinforcement, as a function of time. Using statistics primarily based on the maintenance practices and costs of Michigan DOT in the analysis, it was found that, although CFRP reinforced bridges may be significantly more expensive than steel reinforced bridges initially, the CFRP alternative becomes the least expensive option during the lifetime of the structure. Some specific observations are:

1. It was found that traffic volume has a significant impact on LCC, as well as the cost effectiveness of CFRP reinforced bridges relative to steel reinforced bridges. Use of CFRP reinforcing demonstrated the most reduction in LCC from steel reinforced bridges in areas of high traffic volume.
2. Use of CFRP reinforcement had lower LCC in AASHTO beam bridges as compared to box beam bridges. Therefore, the most effective use of CFRP reinforcement would be for an AASHTO beam bridge in a high traffic volume area.
3. Although more expensive initially, the use of CFRP reinforcement has the potential to achieve significant reductions in LCC, with the steel alternatives exceeding the CFRP bridge LCC from 53-205% at year 100. The break-even year ranged from 6-40, while CFRP was found highly likely to be the least expensive alternative (≥ 0.95) from years 23-77 after initial construction, depending on the bridge case considered.

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Table 1. Below Bridge Initial AADT

	Below Bridge Traffic Volume*		
Bridge Span	Low	Medium	High
Short	10,000	30,000	--
Medium	20,000	60,000	100,000
Long	--	100,000	140,000

*Max. AADT is 120,000; 200,000; and 250,000 for low, medium, and high traffic volumes, respectively

Table 2. Activity Timing Random Variables for Steel-Reinforced Bridges

RV	Description	Mean (yrs)	Initialized From (i.c. = initial construction)	COV
T1	Superstructure replacement	65	i.c.; previous T1	0.13
T2	Deck and beam replacement	40	i.c.; T1**	0.08
T3	Deck overlay, beam end repair	20	i.c.; T1; T2; previous T3***	0.29
T4	Deck patch	8	i.c.; T1; T2; T3; previous T4****	0.21
T5	Cathodic protection*	25	i.c.; T1; T2	0.15

*for black steel only
**no closer to T1 than 10 years
***no closer to T1 or T2 than 10 years
****no closer to T1, T2, T3, or T4 than 2 years

Table 3. Agency Cost Random Variables

RV	Description	Mean ($)			COV
		BS	EC	CFRP	
AC1**	Bridge construction	650700*	559000	951500	0.20
AC2	Deck patch	23190	23190	--	0.40
AC3	Deck overlay	231700	231700	231700	0.40
AC4	Deck replacement	544700	544700	579500	0.20
AC5	Beam end repair	17600	17600	--	0.60
AC6	Beam replacement	128900	128900	--	0.20
AC7	Cathodic protection maintenance	1140	--	--	0.40
AC8	Cathodic protection upgrade	48000	--	--	0.40
AC9	Superstructure demolition	110400	110400	110400	0.20
AC10	Routine inspection	460	460	460	0
AC11	Detailed inspection	8530	8530	8530	0

*BS is more expensive initially than EC due to the initial cathodic protection.
**Cost given for box beam bridge. For the AASHTO beam bridge, the mean of AC1 has values of 684000, 587500, and 740700 for BS, EC, and CFRP, respectively.

Table 4. User Cost Parameters

Parameter	Value
L*	0.5 - 2 miles
N*	4 hours - 5 months
S_n (on bridge)	45mph
S_a (on bridge)	30mph
S_n (below bridge)	70mph
S_a (below bridge)	45mph
w	$13.61
r	$11.22
c_a	$99,560
A_a	2.58%
A_n	1.56%

*L and N vary from low (routine inspection) to high (superstructure replacement) values, based on the activity.

Table 5. User Cost Random Variables

RV	Description	Mean ($)*
UC1	Deck patch	107000
UC2	Deck shallow overlay	254300
UC3	Deck replacement	406400
UC4	Superstructure replacement	677300
UC5	Cathodic protection maintenance	277
UC6	Cathodic protection upgrade	1385
UC7	Routine inspection	55
UC8	Detailed inspection	4792

*COV varies in the analysis and is a function of COVs for travel time cost (0.12), operating cost (0.18), and crash cost (0.13).

Table 6. Mean LCCA Results (millions of dollars)

Bridge Type	Reinforcement	Initial Cost	LCC, HH	LCC, ML	LCC, LL
Box Beam	BS	0.70*	5.98	3.52	2.23
	EC	0.61	5.63	3.34	2.04
	CFRP	0.98	2.23	1.69	1.33
AASHTO Beam	BS	0.71*	5.39	3.24	2.06
	EC	0.61	5.04	3.05	1.86
	CFRP	0.75	1.77	1.33	1.02

*Here the cost of initial cathodic protection (0.10 million) is added to the initial construction cost of BS; removing this cost would result in the initial cost of BS being cheaper than EC.

Table 7. Mean LCC Breakdown for Box Beam Bridges (millions of dollars)

Cost Item	BS	EC	CFRP
Initial Construction	0.60	0.61	0.98
Initial Cathodic Protection	0.10	----	----
Routine Inspection	0.02	0.02	----
Detailed Inspection	0.29	0.29	0.15
Deck Patch	0.23	0.23	----
Deck Overlay	1.85	1.85	0.60
Deck Replacement	1.32	1.32	0.51
Beam End Repair	0.01	0.01	----
Beam Replacement	0.04	0.04	----
Cathodic Protection Maint.	0.19	----	----
Cathodic Protection Upgrade	0.06	----	----
Superstructure Demolition	0.02	0.02	----
Superstructure Replacement	1.23	1.24	----
Total Agency Cost	1.43	1.25	1.11
Total User Cost	4.55	4.38	1.12
Total Life-Cycle Cost	5.98	5.63	2.23

Table 8. LCCA Results Summary

Case	Year when the probability that CFRP costs less is ≥ 0.50				Year when the probability that CFRP costs less is ≥ 0.95			
	Box Beam		AASHTO Beam		Box Beam		AASHTO Beam	
	Black Steel	Epoxy-Coated	Black Steel	Epoxy-Coated	Black Steel	Epoxy-Coated	Black Steel	Epoxy-Coated
LL	35	40	12	20	77*	(76)**	43	59
ML	20	21	11	16	42	44	33	39
HH	15	18	6	13	29	38	23	26

*After year 77, the probability that CFRP costs less decreases steadily to 0.91 at year 100.
**The maximum probability that CFRP costs less is 0.89 at year 76, and decreases steadily to 0.82 at year 100

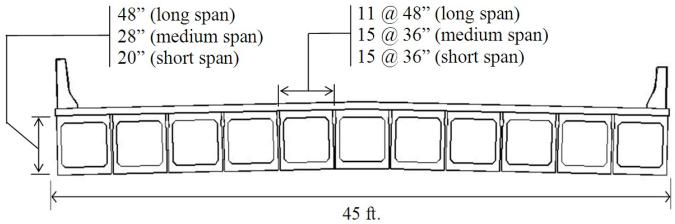

Figure 1. Medium Span Box Beam Bridge Cross-Section.

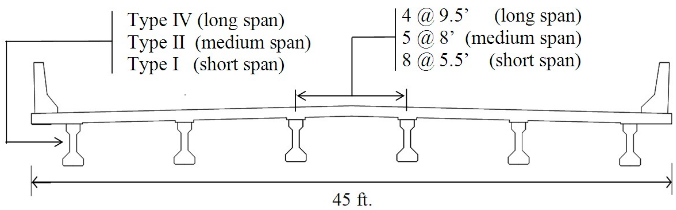

Figure 2. Medium Span AASHTO Beam Bridge Cross-Section

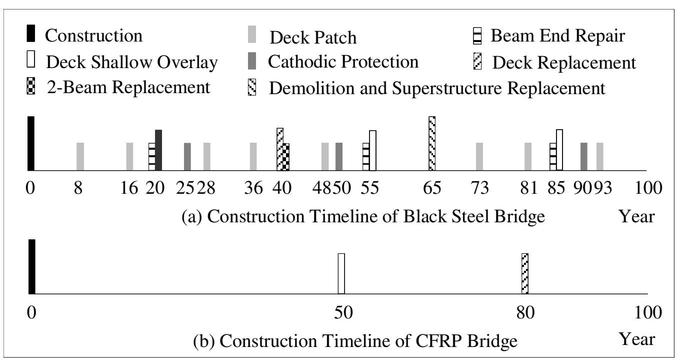

Figure 3. Activity Timeline – Mean Times (inspections not shown)

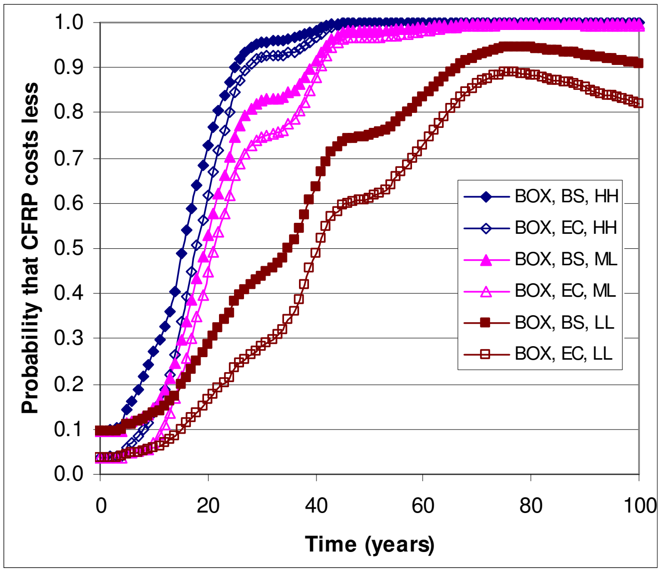

Figure 4. Results for Box Beam Bridges

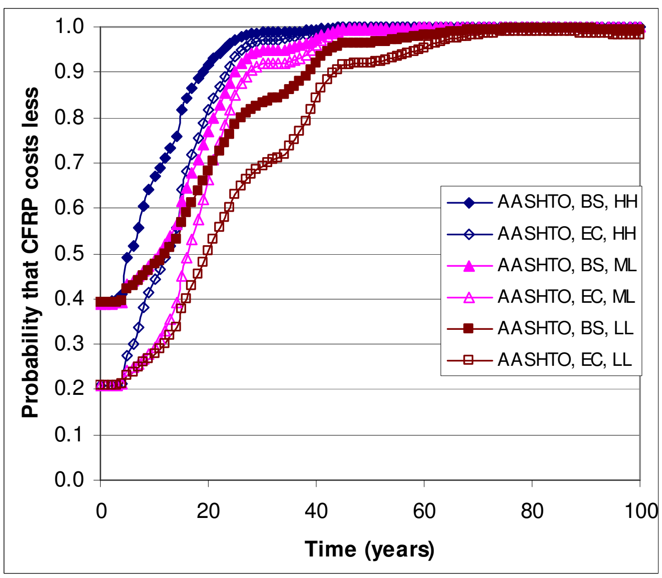

Figure 5. Results for AASHTO Beam Bridges