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Abstract: Markovian transition probability matrices employing condition states are often used in bridge management systems to determine optimal intervention strategies. This approach assumes a constant deterioration matrix throughout the entire analysis period. However, in order to adequately model and evaluate intervention options such as fiber-reinforced polymer (FRP) strengthening, it is necessary to model the impact of the intervention on the deterioration rate. This paper presents a Markovian based approach to model interventions that impact deteriorating rates. A model employing this approach is proposed. A methodology to determine the optimal intervention strategy based on steady state probabilities is then presented. The proposed model and methodology are illustrated in an evaluation of intervention options for a concrete girder bridge.

Keywords: FRP-strengthening, concrete, deterioration modelling, Markovian modelling, optimal intervention strategy.

1. Introduction

Bridge managers often use bridge management systems (BMSs) to determine optimal interventions strategies (OISs) to be implemented on bridges so that these structures will continue to provide adequate level of service. Most advanced of these BMSs often use condition state (CS) based Markovian modeling approach to determine the OISs [1,2,3]. A common assumption made by the modeling approach of these BMSs, is that the deterioration matrix will remain unchanged under the interventions [1,2,3]. The modeling approach used by the BMSs is sufficient for modeling traditional intervention actions, such as replacement or “patching” of bridge elements, where the intervention can be assumed to change the CS, but not the deterioration rate. These methodologies are inadequate, however, for evaluating certain intervention actions (e.g. FRP strengthening), which can also influence the deterioration rate of the element.

External strengthening of concrete structures using fiber-reinforced polymer (FRP) composite materials has become increasingly popular in recent years. The high strength-to-weight ratio of FRPs, which helps to minimize the labor cost associated with the strengthening of bridge elements, has made this method attractive to infrastructure engineers. However, if these types of novel interventions are to be considered in existing BMSs, a new methodology which could consider the change in deterioration matrix, as a result of the intervention action is necessary.

This paper presents a methodology to determine the OISs considering intervention actions that result in deterioration rate changes. This methodology employs a modified CS based transition probability matrix to model deterioration, allowing changes in the deterioration rate to occur during the analysis period as a result of the modeled intervention strategies (ISs). Steady state Markovian properties are used to determine the OIS. Finally, the proposed methodology is illustrated using a case study of a concrete girder bridge retrofitted by FRP strengthening.

2. Proposed model

2.1. Condition based transition probability matrix for deterioration modelling

A typical transition probability matrix of an element with n CSs can be written as:

$$P_e = p_{ij}^e = \begin{bmatrix} p_{11}^e & p_{12}^e & \cdots & p_{1n}^e \\ 0 & p_{22}^e & \cdots & p_{2n}^e \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & 1 \end{bmatrix} \tag{1}$$

with:

$$\begin{cases} \sum_{j=1}^{n} p_{ij}^e = 1 & \forall\, i,\, j \\ p_{ij}^e = 0 \text{ when } (i > j) \end{cases} \tag{2}$$

where index e denotes the element of concern, and n is the number of CSs for element e. In such a transition matrix the worst (i.e. highest) CS is defined as the CS where the element performance becomes inadequate or the element becomes functionally obsolete.

A possible intervention for a concrete beam is to be strengthened using externally bonded FRP sheets. After such an intervention, the critical deterioration mechanism of such a strengthened beam becomes the interfacial damage of the FRP-concrete interface [4,5], which will have a different deterioration rate than the original concrete beam. The basic difference in such a case is that when an intervention is carried out, the deterioration will follow a new path. In order to model this, when an intervention is carried out in CS *j*, the element can be assumed to transit to a new deterioration matrix, which has the transition probabilities according to the new element deterioration rate. Under such situation two different deterioration matrices could be defined for; (1) original element (i.e. concrete element, denoted by *e*=1) with *N* number of CSs (i.e. *n*=*N*), and (2) new element (i.e. FRP strengthened concrete element denoted by *e*=2) with *K* number of CSs (i.e. *n*=*K*).

2.2. Condition based transition probability matrix for improvement modelling

The interventions maybe carried out on element 1 (i.e. concrete beam) or on element 2 (i.e. FRP strengthened concrete beam). If the interventions were carried out on element 1, depending on the intervention action CS may be transit to another CS of element 1 or CS may transit to a CS of element 2. Similarly, if the interventions were carried out on element 2, depending on the intervention action CS may transit either to CSs of element 1 or CSs of element 2. The transition probabilities due to the execution of intervention, therefore could be written as:

$$R_e = {}^{e}r_{ij}^{e'} = \begin{bmatrix} {}^{e}r_{11}^{1} & {}^{e}r_{12}^{1} & \cdots & {}^{e}r_{1N}^{1} & {}^{e}r_{11}^{2} & {}^{e}r_{12}^{2} & \cdots & {}^{e}r_{1K}^{2} \\ {}^{e}r_{21}^{1} & {}^{e}r_{22}^{1} & \cdots & {}^{e}r_{2N}^{1} & {}^{e}r_{21}^{2} & {}^{e}r_{22}^{2} & \cdots & {}^{e}r_{2K}^{2} \\ \vdots & \vdots & \ddots & \vdots & \vdots & \vdots & \ddots & \vdots \\ {}^{e}r_{n1}^{1} & {}^{e}r_{n2}^{1} & \cdots & {}^{e}r_{nN}^{1} & {}^{e}r_{n1}^{2} & {}^{e}r_{n2}^{2} & \cdots & {}^{e}r_{nK}^{2} \end{bmatrix} \tag{3}$$

with:

$$\begin{cases} {}^{e}r_{ij}^{2} = 0 \quad \forall\, i \quad if \quad Int = 1 \\ {}^{e}r_{ij}^{1} = 0 \quad \forall\, i \quad if \quad Int = 2 \\ \sum_{j=1}^{N} {}^{e}r_{ij}^{1} + \sum_{j=1}^{K} {}^{e}r_{ij}^{2} = 1 \quad \forall\, i = i' \\ {}^{e}r_{ij}^{2} = {}^{e}r_{ij}^{2} = 0 \qquad \forall\, i \neq i' \end{cases} \tag{4}$$

where *Int* denotes the type of intervention to be chosen, i.e. if *Int* = 1, element similar to the element 1 will be resulted, and if *Int* = 2, then an element similar to the element 2 will be resulted. If the interventions are carried on element 1, *e*=1 and *n*=*N* and if the interventions are carried out on element 2, *e*=2 and *n*=*K*.

A combined intervention effectiveness matrix for both elements can be written as:

$$\overline{R}_c = \overline{r}_{ij}^c = \begin{bmatrix} \overline{r}_{1,1}^c & \cdots & \overline{r}_{1,N}^c & \overline{r}_{1,N+1}^c & \cdots & \overline{r}_{1,N+K}^c \\ \vdots & \ddots & \vdots & \vdots & \ddots & \vdots \\ \overline{r}_{N,1}^c & \cdots & \overline{r}_{N,N}^c & \overline{r}_{N,N+1}^c & \cdots & \overline{r}_{N,N+K}^c \\ \overline{r}_{N+1,1}^c & \cdots & \overline{r}_{N+1,N}^c & \overline{r}_{N+1,N+1}^c & \cdots & \overline{r}_{N+1,N+K}^c \\ \vdots & \ddots & \vdots & \vdots & \ddots & \vdots \\ \overline{r}_{N+K,1}^c & \cdots & \overline{r}_{N+K,N}^c & \overline{r}_{N+K,N+1}^c & \cdots & \overline{r}_{N+K,N+K}^c \end{bmatrix} \quad (5)$$

with:

$$\begin{cases} \overline{r}_{ij}^C = {}^1r_{ij}^{e'} & \forall i \le N \\ \overline{r}_{ij}^C = {}^2r_{ij}^{e'} & \forall N+1 \le i \end{cases} \quad (6)$$

2.3. Combined deterioration intervention matrix

The combined deterioration intervention matrix can be written as:

$$\overline{Q}_C = \overline{q}_{ij}^C = \begin{cases} \hat{p}_{ij}^1 + \overline{r}_{ij}^c & \forall i = 1,2,\ldots N,\ j = 1,2\ldots.N+K \\ \hat{p}_{ij}^2 + \overline{r}_{ij}^c & \forall i = N+1, N+2,\ldots N+K,\ j = 1,2\ldots.N+K \end{cases} \quad (7)$$

with:

$$\begin{cases} \hat{p}_{ij}^1 = p_{ij}^1 & \forall i \ne \grave{\imath}', i \le N,\ j \le N \\ \hat{p}_{ij}^1 = 0 & \forall j \ge N+1 \\ \hat{p}_{ij}^1 = \hat{p}_{ij}^2 = 0 & \forall i = \grave{\imath}' \\ \hat{p}_{ij}^2 = p_{ij}^2 & \forall i \ne \grave{\imath}', i \ge N+1,\ j \ge N+1 \\ \hat{p}_{ij}^2 = 0 & \forall j \le N \end{cases} \quad (8)$$

The CS of the element in any given year can be obtained by:

$$\Pi_C(t) = \Pi_C(0)\left(\overline{Q}_C\right)^t \quad (9)$$

where $\Pi_C(0) = \left\{\pi_1^c(0) \quad \pi_2^c(0) \quad \cdots \quad \pi_{N+K}^c(0)\right\}$ is the CS distribution of the element at $t = 0$.

2.4. Determination of the optimal intervention strategy

Using the above combined deterioration intervention matrix, the total life cycle impacts (LCIs) of each IS can be calculated, thus the OIS, which results in the lowest total LCI, can be determined. However, when there are many ISs, the calculation procedure could be tedious. An alternative to determine the OISs is proposed in this section using the steady state properties, as being done in many existing BMSs [2,3].

The total expected impacts of an IS per time interval under steady state (stationary transition) conditions can be calculated as:

$$E(V) = \sum_{\forall i=i' \le N} \sum_{j=1}^{N} \tilde{\pi}_{j,I}^e \left(\sum_{a=1}^{A} c_{a,i}^{e,I}\right) + \sum_{\forall i=i' \ge N+1} \sum_{j=1}^{K} \tilde{\pi}_{j+N,I}^e \left(\sum_{a=1}^{A} c_{a,i}^{e,I}\right) + \sum_{j=1}^{N+K} \tilde{\pi}_{j,I}^e \sum_{a=1}^{A} c_{a,j}^{e,D} \frac{(1-d)}{d_T} \quad (10)$$

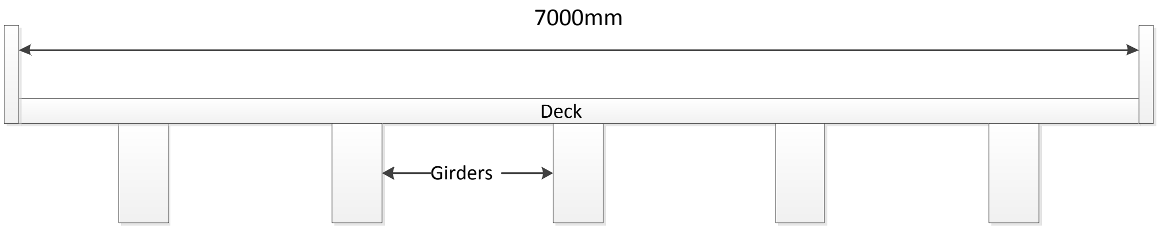
where $\tilde{\pi}^{e}_{j,I}$ is the steady state probability [6] of element *e* in CS *j* under IS *I*, $c^{e,I}_{a,i}$ is the value of impact *a* in carrying out intervention *i* on element *e*, $c^{e,D}_{a,j}$ is the value of impact *a* when the element is in operation and in CS *j*, d_T is the length of the time interval *t* in days, and *d* is the expected number of days the bridge is closed for interventions during a time interval given by:

$$d = \sum_{\forall i=i' \leq N} \sum_{j=1}^{N} \pi^{e}_{j,I} d^{e,I}_{i} + \sum_{\forall i=i' \geq N+1} \sum_{j=1}^{K} \pi^{e}_{j+N,I} d^{e,I}_{i} \tag{11}$$

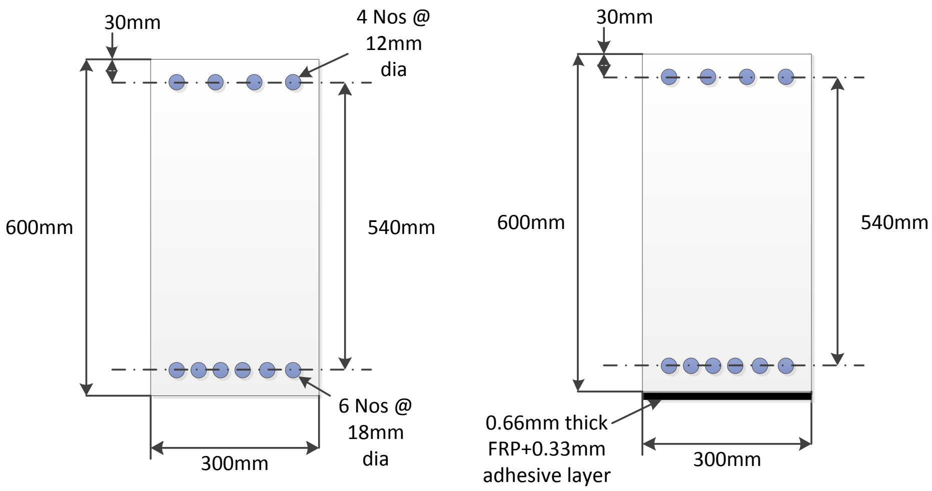
The OIS is selected as the IS resulting in the lowest expected impacts per time interval.

3. Example

In order to demonstrate the application of the above methodology, the life cycle performance of a short span concrete girder bridge with one rehabilitation option being FRP strengthening was studied in this research. The cross sections of the bridge, the reinforced concrete beam, and the FRP strengthened beam are given in Figures 1a, 1b, and 1c, respectively. The bridge considered in this example has a span of 5 m, and is 7 m wide.



(a) Cross section of the bridge



(b) Concrete beam cross section **(c) FRP-strengthened concrete beam cross section**

Figure 1. Cross sections of the concrete and FRP-strengthened concrete beams.

3.1. CS definitions

Typically, the CSs of concrete elements subjected to reinforcement corrosion are defined in terms of reinforcement section loss [2]. Similarly, in the current study the CSs for the concrete beam (called CCS hereafter) were defined based on the reinforcement section loss (Table 1). As the main deterioration of the FRP strengthened concrete beam is the bond degradation, the CSs for the FRP strengthened concrete beam (called FCS hereafter) were defined using the bond strength loss (Table 1). The FCSs were set so that, the worst FCS (i.e. FCS3) gives equal performance to the worst CCS (i.e.CCS5).

Table 1. CS description for concrete beams

Beam type	Condition state	Description	Intervention action	CS after the intervention
Concrete beam	CCS1	as new, no corrosion	do nothing	
	CCS2	corrosion initiation, <2% thickness loss	do nothing	
	CCS3	moderate corrosion, <6% thickness loss	do nothing	
	CCS4	high corrosion, <12% thickness loss	do nothing	
	CCS5	severe corrosion, ≥12% thickness loss	rehabilitation/ replacement	FCS1/CCS1
FRP-concrete beam	FCS1	as new, loss in bond strength <10%	do nothing	
	FCS2	loss in bond strength 10-25%	do nothing	
	FCS3	loss in bond strength ≥25%	replacement	CCS1

3.2. Deterioration matrix for concrete beams

The CCSs of the RC beam are defined based on the corrosion of the reinforcement. The corrosion of the reinforcement can be modeled stochastically using two-part corrosion model. Such a corrosion model is defined in two parts, namely, time until corrosion initiation, and time of corrosion propagation. In order to determine the corrosion initiation time, it is necessary to calculate the chloride concentration at the reinforcement location at any given time. At time $s(t)$ (in seconds), the chloride concentration at a depth *x* can be given from a modified formula, based on the original derivation of Gjrv and Vennesland [7], as:

$$C(x, s(t)) = C_0 + (C_i - C_0)\left\{ erf\left(\frac{x}{2\sqrt{D_C \cdot s(t)}} \right) \right\} \qquad (12)$$

where *D* is the chloride diffusion coefficient, C_0 is the equilibrium chloride concentration on the concrete surface as % by weight of cement, C_i is the initial chloride concentration, and *erf* is the error function. It is assumed that a corrosion process is initiated when the chloride concentration at the reinforcement location reaches a certain critical chloride corrosion threshold value C_{cr}. The critical chloride threshold depends on the type of concrete and several other factors [8].

The corrosion process after corrosion initiation is very difficult to model. Different models exist with different levels of sophistication to predict the corrosion propagation process [8,9]. The simplest model is to assume that the diameter $\phi(t)$ of the reinforcement bars at the time t_c (after corrosion initiation) is modeled by [9]:

$$\phi(t) = \phi_0 - \lambda i_{corr}(t_c) \qquad (13)$$

where ϕ_0 is the initial diameter, λ is a factor to convert average corrosion densities to average penetration rates (2.3294·10^{-3} mm/(mA/m^2)), and i_{corr} is the rate of corrosion.

Using the above models, a Monte Carlo (MC) simulation was carried out to determine the residual reinforcement bar diameter of the concrete beam over the life time (i.e. 75 years). The distribution properties of the parameters used in the MC simulation are given in Table 2. Based on the residual diameter values of the reinforcement, the CCS distributions were identified. Using these CCS distributions, the transition probabilities of the transition matrix were calculated using a computer program coded in Matlab [10]. The method used for calculating the transition probabilities is the same as used in Fu and Devaraj [11]. The resulting transition probability matrix for a reinforced concrete beam is given in Table 3.

Table 2. Distribution properties of different parameters for concrete beam

Parameter	Units	Mean	COV %	Distribution type
C_{cr}		0.2	50	Normal
C_0		5	20	Lognormal
C_i		0.01	100	Normal
D_c	m/s^2	$4.00\cdot10^{-12}$	70	Normal
f_c	MPa	35	18	Lognormal
f_y	MPa	450	12.5	Lognormal
h	mm	600	3	Normal
b	mm	300	3	Normal
d_1	mm	30	3	Normal
$\phi_{st-top}, \phi_{st-bottom}$	mm	12,18	1.5	Normal

COV: coefficient of variation

Table 3. Deterioration matrix for concrete beam

	Year *(t + 1*)				
Year (*t*)	*CS1*	*CS2*	*CS3*	*CS4*	*CS5*
CS1	0.918	0.082	0.000	0.000	0.000
CS2	0.000	0.620	0.380	0.000	0.000
CS3	0.000	0.000	0.841	0.159	0.000
CS4	0.000	0.000	0.000	0.894	0.106
CS5	0.000	0.000	0.000	0.000	1.000

3.3. Deterioration matrix for FRP strengthened concrete beams

Durability of the FRP strengthened systems are often affected by the moisture conditions. The studies have shown that the interfacial fracture energy significantly reduces with the increasing moisture content [12,13]. Tuakta and Büyüköztürk [14] proposed the following equation to determine the interfacial fracture energy under moisture cycles:

$$G_{fr}(t) = G_{f0} - \sum_{t=0}^{t} N_f(t) \cdot q \left(C_{int} / C_{th} \right)^n \tag{14}$$

where $G_{fr}(t)$ is the residual fracture energy at time *t*, G_{f0} is the initial fracture energy, $N_f(t)$ is the number of moisture cycles at time *t*, C_{int} is the intermediate moisture content, C_{th} is the threshold moisture content, and q and n are coefficients which are typically determined by experimental results. The interfacial fracture energy is then related to the bonding strength as:

$$f_{frp,d}(t) = E_{frp}\varepsilon_{frp,d} = 0.9\beta_L \sqrt{\frac{2E_{frp}G_{fr}(t)}{t_{frp}}} \tag{15}$$

In the current study, the coefficients *q* and *n* were set to 224N/m and 0.83, respectively. The initial fracture energy G_{f0}, number of moisture cycles per year N_f, and $\left(C_{int} / C_{th} \right)$ ratio were modeled as stochastic parameters and the assumed distribution properties are given in Table 4. It should be noted that the values selected here are assumed for an abstract condition and not intended for a real life-cycle cost comparison of FRP-strengthened beam. Only an illustration of the methodology was intended. For accurate results, more sophisticated analysis involving moisture diffusion analysis [15] and better experimental results for determining coefficients are necessary.

Table 4. Assumed distribution properties for G_{f0}, N_f, and $\left(C_{int}/C_{th}\right)$

Parameter	Units	Mean	COV %	Distribution type
G_{f0}	N/m	534	18	Log normal
N_f (annual*)		1.00	35	Lognormal
$\left(C_{int}/C_{th}\right)$		0.004	15	Lognormal
E_{frp}	MPa	150000	10	Lognormal
t_{frp}	mm	0.66	1.5	Lognormal

*Based on 6 hour 50 mm rainfall

Using the above models, a MC simulation was carried out to determine the residual bond strength of the FRP-concrete bond joint over its life time. Based on the residual bond strength values of the reinforcement, the FCS distributions were identified. Using these FCS distributions, the transition probabilities of the transition matrix were calculated using a computer program similar to the one used for the concrete beam. The resulting transition probability matrix for FRP strengthened beam is given in Table 5.

Table 5. deterioration matrix for FRP strengthened concrete beam

		Year (*t+1*)		
Year (*t*)		*FCS1*	*FCS2*	*FCS3*
	FCS1	0.9817	0.0183	0.0000
	FCS2	0.0000	0.9878	0.0122
	FCS3	0.0000	0.0000	1.0000

3.4. Intervention options

Two different intervention strategies were considered in this study, i.e.:

1) IS1- replacement of the beam with a similar concrete beam in CCS5, and
2) IS2- FRP strengthening in CCS5 and replacement with a concrete beam in FCS3.

When replaced with a concrete beam, it was assumed that the original CS will be restored, i.e. CCS1 of the un-strengthened RC beam. When FRP strengthening was carried out, it was assumed the CS will be improved to FCS1 for the FRP strengthened beam.

With the above assumptions, the resulting intervention effectiveness matrices for the two different ISs, i.e. IS1, and IS2, are given in Table 6.

Table 6. Intervention effectiveness matrix for IS1 and IS2

Intervention strategy		After the intervention								
	Before the intervention	CS	*CCS1*	*CCS2*	*CCS3*	*CCS4*	*CCS5*	*FCS1*	*FCS2*	*FCS3*
IS1		*CCS5*	1	0	0	0	0	0	0	0
IS2		*CCS5*	0	0	0	0	0	1	0	0
		FCS3	1	0	0	0	0	0	0	0

The combined deterioration-intervention matrices for each IS can be calculated from Equations 3 to 9.

3.4. Intervention options

The impacts considered in this example includes both owner and public impacts. Further subdivision of these impacts can be found in numerous references [16]. However, as only an illustration was intended, such detailed impact calculations were not carried out. The considered impacts were divided into during interventions and during operations. The impacts during the interventions are listed in Table 7.

Table 7. Impacts during the interventions

Event	Number of bridge closure days	Total owner impacts (mu)	Total public impacts (mu)
Concrete beam replacement	18	2200	180
FRP repair	2	760	20

mu: monetary units

The impacts during bridge operation include the owner impacts (mainly the routine maintenance costs), and public impacts (i.e. the impacts to the public due to the usage of the bridge). The values of the public impacts were considered to vary with the condition of the bridge [17]. The assumed values in this example are given in Table 8.

Table 8. Impacts during the operations

Cost type	CS							
	CS1	CS2	CS3	CS4	CS5	FCS1	FCS2	FCS3
Annual maintenance cost (mu)	180	180	180	180	180	180	180	180
Annual public cost (mu)	1750	1825	1900	2000	2150	1750	1850	2150

mu-monetary units

3.4. Optimal intervention strategy

The OIS was selected as the IS results in the minimum life-cycle cost based on the methodology described in Section 2.4. The calculated steady state probabilities for each IS are shown in Table 9.

Table 9. Steady state probabilities for each IS

Intervention Strategy	Steady State probabilities							
	CS1	CS2	CS3	CS4	CS5	FCS1	FCS2	FCS3
IS1	0.385	0.084	0.200	0.300	0.032	0.000	0.000	0.000
IS2	0.072	0.016	0.037	0.056	0.006	0.323	0.485	0.006

The calculated long term expected annual impacts of each IS are shown in Table 10.

Table 20. Long term expected annual impacts

Intervention strategy	Annual impacts during interventions (mu)		Annual impacts during operations (mu)		Total annual impacts (mu)
	Owner	Public	Owner	Public	
IS1	69.70	5.70	180.00	1870.91	2126.32
IS2	17.48	1.18	180.00	1823.28	2021.94

mu-monetary units

It can be seen in Table 10 that the IS2 (i.e. FRP strengthening) would result in lower impacts than the IS1 (i.e. concrete beam replacement). The highest percentage difference was observed in the annual impacts during the interventions. Significant reduction in both the owner and public impacts during the interventions is due to less closure days for the FRP strengthening option compared to the concrete beam replacement option.

5. Summary and conclusions

This paper presents a methodology for evaluating the life-cycle impacts of intervention strategies, which considers changing deterioration rates. The methodology was developed based on the Markovian approach commonly used in existing bridge management systems. Based on the steady state properties, a simplified method was proposed to determine the optimal intervention strategies. The proposed methodology is applied in an LCCA framework that considers impacts to different stakeholders both during and between the interventions.

The proposed methodology is demonstrated for a concrete girder bridge, where one of the intervention options is FRP strengthening. The example uses existing corrosion models, together with probability distributions of relevant parameters, to model the reinforcement corrosion stochastically. A similar approach is used to model the FRP bond strength degradation. Using these deterioration models, the transition probabilities are determined for plain and FRP strengthened concrete girders. The transition probability matrices are then used to evaluate the life-cycle impacts of each intervention strategy and determine the optimal intervention strategy.

The results show that under the conditions used in the example, FRP strengthening resulted in lower negative life-cycle impacts compared to girder replacement. The highest annual impact reduction due to FRP strengthening option was observed in the impacts during the interventions. Both the owner and public benefitted due to FRP strengthening option compared to the concrete girder replacement option.

The example is intended only to illustrate the methodology, thus the findings of this example should not be extended to general application. More accurate deterioration modeling of the FRP strengthened girders as well as more detailed impact modeling is necessary for accurate life-cycle impact evaluation of the FRP strengthening. Nevertheless, the proposed methodology is seen to provide an efficient means of considering the changing deterioration rates in evaluating the life-cycle impacts of intervention strategies.

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