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RELIABILITY-BASED LIFE-CYCLE COST OPTIMIZATION OF BRIDGE PIERS EXPOSED TO EARTHQUAKE AND NON-UNIFORM CORROSION

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Abstract

Chloride-induced reinforcement corrosion is the main cause of the performance deterioration of the reinforced concrete (RC) coastal bridges. Regular maintenance and repair activities are required for the corroded bridges, which will lead to significant cost. Aiming at this problem, there have been many studies on the life-cycle cost (LCC) analysis of structures over the past half-century. However, the effects of design materials on the LCC of RC structures considering the risk of corrosion and earthquake have not been studied sufficiently. For coastal bridges located in earthquake-prone areas, the LCC is more than that of urban bridges due to a more aggressive environment. However, the relative research is still extremely rare.

In this paper, a reliability-based design optimization (RBDO) procedure is developed for the design of the RC bridge pier. To compare the cost-effectiveness of the design materials, five groups of commonly-used design materials are used. In particular, an RC bridge pier is selected as a case study. Firstly, the RBDO procedure is developed for the optimization design of bridge piers. Then, the bridge pier is designed based on the procedure, given the corresponding constraints. The initial construction cost ($C_{initial}$) and LCC are taken as the optimization objectives respectively in the design procedure to compare the difference in the design results. Thirdly, reliability analysis of the piers designed with each group of materials considering the major uncertainties is carried out to obtain the time-dependent reliability in terms of the ultimate and serviceability performances. Due to the special corrosion environment, the pier is divided into submerged zone, splash and tidal zone, and atmospheric zone along its height in terms of the risk of corrosion. Corrosion in the three zones is considered separately in the design. Next, the repair time of the piers is predicted based on the time-dependent reliability indexes. Finally, the time-dependent LCCs for both the piers with $C_{initial}$ and LCC as optimization objectives are obtained to select the optimal design materials for coastal bridge piers under corrosive environment.

Keywords: RBDO; LCC; earthquake; non-uniform corrosion; coastal bridge pier

1. Introduction

Coastal bridges, as an important part of the transportation system, are exposed to the chloride ions. Chloride-induced corrosion of the reinforcement is one of the major causes of deterioration of the bridges [1], which will reduce the performance of the bridges, especially the capacity and serviceability of the bridge pier. On the other hand, the corrosion risk at different regions of the coastal bridge pier is different, which will lead to non-uniform corrosion along its elevation [2]. For the bridges in the earthquake-prone region, corrosion will make the bridges more vulnerable. However, the current design specification does not take the chloride-induced degradation effect into consideration when designing new bridges.

In order to improve the cost-effectiveness and ensure the safety of the structures and public, optimization design can be adopted in the design process. Sajedi and Huang [3] introduced an RBDO procedure for the design of RC bridge beam and illustrated to an RC girder beam exposed to deicing salt and compared the LCC of different types of materials. However, there are few studies on bridge piers.

The main objective of this paper is to develop the RBDO procedure for the design of coastal bridge piers exposed to chlorides and earthquake. The non-uniform corrosion effect will be considered in the design process. Five types of commonly used materials are used to design the same bridge pier to compare their cost-effectiveness. And C_{initial} and LCC are taken as optimization objectives respectively to compare the design results.

2. Corrosion modeling

A protective passive film forms on the reinforcement when placed in the concrete as a result of the high alkaline environment provided by the surrounding concrete. The occurrence of chloride penetrates from the environment will destroy this passive film and lead to the corrosion of the reinforcement. The corrosion process is divided into two phases [3, 4], i.e., corrosion initiation phase and corrosion propagation phase.

During corrosion initiation phase, chlorides, oxygen and water from the external environment penetrate through the concrete cover to the reinforcement surface and cumulate. Chloride diffusion model for uncracked concrete based on Fick's second law [5] is adopted in this paper. It is assumed that the chloride concentration on the concrete surface is constant and the initial chloride concentration in the concrete is zero. Chloride concentration at time t, distance x (to the nearest structure boundary), $C(x, t)$ can be determined by Eq.(1) [6], where C_s is the surface chloride concentration, erf ( ) is error function, and D_c is the diffusion coefficient of the concrete. C_s for the atmospheric zone and splash and tidal zone is 2.95 and 7.35 kg/m^3 respectively [7]. It is worth mentioning that the diffusion model is for the uncracked media. The models account for cracking have been proposed for flexural members are not available for the column, for the axial load will lead to the closure of the cracks, thus the influence of cracking will be diminished [8]. When the chloride concentration reaches the critical chloride threshold (Cl_{th}) of the reinforcement, corrosion starts. Set x = C_{b}, which is the distance between the reinforcement and the nearest surface of the concrete, and $C(x\ ,t)=Cl_{\mathrm{th}}$ in Eq.(1), then the corrosion initiation time, t_{in}, can be obtained by solving Eq.(1), as shown in Eq. (2).

$$C(x,t)=C_{\mathrm{s}}\left\{1-erf\left(\frac{x}{2\sqrt{D_{\mathrm{c}}t}}\right)\right\} \tag{1}$$

$$t_{\mathrm{in}}=\frac{C_{\mathrm{b}}^{2}}{4D_{\mathrm{c}}\left[erf^{-1}\left(1-\frac{Cl_{\mathrm{th}}}{C_{\mathrm{s}}}\right)\right]^{2}} \tag{2}$$

The corrosion propagation phase starts when the passive film is broken down. Corrosion of reinforcement causes extensive damage to the structures. On the one hand, corrosion will result in the loss of strength and cross-section area of the reinforcement, and the diameter and yield strength of the corroded reinforcement at time t, $d(t)$ and $f_y(t)$, can be estimated through Eqs. (3) and (4) respectively [9], where d_0 and f_{y0} refers to the diameter and yield strength of the intact reinforcement respectively, $Q(t)$ refers to the percentage mass loss of the rebar, and i_{corr} refers to the corrosion current density. The corrosion product has a property of expansion, which will cause expansion force between the reinforcement and concrete cover, and then lead to cracking, spalling and delamination. The surface crack width results from corrosion can be determined by Eq. (6), where α_d is the volume expansion rate between the corrosion product and the intact steel, which is usually assumed to be 2.0 [10], and $t_{cr} = t_{in} + t_{cro}$ refers to the initial crack time of the concrete cover, in which t_{cro} is the time between corrosion initiation cracking and can be calculated by Eqs. (7) and (8).

$$d(t)=\sqrt{1-0.01Q(t)}d_0 \tag{3}$$

$$f_y(t)=\left[1-0.005Q(t)\right]f_{y0} \tag{4}$$

$$Q(t)=\begin{cases} 0 & t<t_{in} \\ 4.6\dfrac{i_{corr}}{d_0}\left(t-t_{in}\right) & t\geq t_{in} \end{cases} \tag{5}$$

$$w_{cr}(t)=\begin{cases} 0 & t<t_{cr} \\ \left[d_0-d(t)\right]\dfrac{(\alpha_d-1)\pi d_0}{\left(\dfrac{0.5d_0}{0.5d_0+C}+1\right)C} & t\geq t_{cr} \end{cases} \tag{6}$$

$$t_{cr0}=\frac{x_{cr}}{11.6i_{corr}} \tag{7}$$

$$x_{cr}=1.25C \tag{8}$$

3. Seismicity

Corrosion makes the structures more vulnerable to other hazards, like earthquakes or collisions, during their design life. To ensure the safety of the bridge structures and public, it is, therefore, necessary to take the corrosion effects into account when designing a new coastal bridge in earthquake-prone region. When it comes to reliability-based earthquake optimization design, there are two main technological aspects that need to be carefully considered. The first is how to compute the seismic force, the second is the distribution of the seismic force.

Several methods can be used to calculate the seismic demand for bridge structures, such as time-history analysis, design response spectrum method, and power spectrum method. In this paper, the design acceleration spectrum method in [11] is adopted. To simplify the analysis, assuming the seismic force obeys normal distribution with a coefficient of variation (CV) 0.3.

4. LCC optimization design procedure

Chloride-induced corrosion of reinforcement degrades the ultimate and serviceability performance of the structure, then maintenance and repair activities, such as patch and overlay, will be required to retrofit the bridge to prevent failure. Regular maintenance and repair will result in high maintenance costs and thus it is now necessary to design coastal bridge piers from the point of view of LCC optimization. An optimization design procedure proposed by Sajedi and Huang [3] takes LCC as the optimization objective, and is illustrated for the design of an RC girder beam exposed to de-icing salt. The optimization procedure is shown as follows:

$$\begin{cases} \text{minimize} \ \ \text{cost}(\hat{\boldsymbol{y}}) \\ \text{subject to:} \begin{cases} \text{probabilistic constraint:} \ \beta_{\mathrm{i}}(\boldsymbol{x},\boldsymbol{y}) \geq \beta_{\mathrm{TDi}} \\ \text{deterministic constraint:} \ D_{\mathrm{j}} \geq D_{\mathrm{min,j}} \\ \text{ranges of design parameters:} \ y_{\mathrm{k}}^{\mathrm{l}} \leq y_{\mathrm{k}} \leq y_{\mathrm{k}}^{\mathrm{u}} \end{cases} \end{cases} \tag{9}$$

where $\boldsymbol{x}$ is the parameters of environment and the materials, such as the surface chloride concentration and material strength, $\boldsymbol{y}$ is the design parameters of the bridge pier, for instance, the diameter and cover depth of the pier and the amount of reinforcement, y_{k} is the kth parameter of $\boldsymbol{y}$, $y_{\mathrm{k}}^{\mathrm{l}}$ and $y_{\mathrm{k}}^{\mathrm{u}}$ are the lower and up boundary of y_{k} respectively, $\hat{\boldsymbol{y}}$ is the mean value of $\boldsymbol{y}$, β_{i} is the reliability index of the ith limit state, β_{TDi} is the design threshold of β_{i}, D_{j} is the jth deterministic constraint, such as the reinforcement ratio, and $D_{\mathrm{min,j}}$ is the limit value of D_{j}.

Eq. (10) shows the performance function, where $C_{\mathrm{i}}(\boldsymbol{x},t)$ and $D_{\mathrm{i}}(\boldsymbol{x},t)$ refers to the capacity and demand of ith limit state respectively. Based on the distribution parameters of the variables related to the design of the bridge pier and setting the performance function equal to 0, the corresponding reliability index can be obtained by FORM. In this study, the flexural capacity of the pier under constant axial load and cover crack width are taken as the ultimate and serviceability limit state respectively. It is assumed that the limit value of crack width has a mean and standard deviation of 0.5 mm and 0.1 mm respectively.

$$g_{\mathrm{i}}(\boldsymbol{x},t) = C_{\mathrm{i}}(\boldsymbol{x},t) - D_{\mathrm{i}}(\boldsymbol{x},t) \tag{10}$$

5. Life cycle cost analysis (LCCA)

As mentioned before, to improve the cost-effectiveness of the bridge structures, it is meaningful to conduct LCCA. The LCC of bridge structures includes initial construction cost (C_{initial}), routine inspection cost, maintenance cost, repair cost, user cost and failure cost. It is worth mentioning that the main purpose of this study is to compare the cost-effectiveness of the bridge pier designed with different materials. To simplify the optimization design process, it is assumed that the user cost and failure cost are constant when the bridge pier is designed with different materials. Thus, the LCC in this study only includes C_{initial}, routine inspection cost, maintenance, and repair cost. Then the LCC in the form of present value can be calculated by Eq. (11), where T is the design service life of the structure, C_{i} is the expected cost incurred in the ith year, and r is the discount rate assumed as 0.02 [12].

$$\mathrm{PVC} = C_{\mathrm{initial}} + \sum_{i=1}^{T} \frac{C_{\mathrm{i}}}{(1+r)^{\mathrm{i}}} \tag{11}$$

Assuming the labor cost is included in the material price, thus C_{initial} can be calculated by the amount of material used and the price of the materials. And the maintenance and repair cost is related to the surface area of the bridge pier and the corresponding unit price. The conducted time for the preventive measurements adopted in this study is shown in Table 1. Patch and overlay are used for the repair of the serviceability and capacity of the pier. Patch is to replace ten percent of the cover of the corresponding zone, and is conducted when the serviceability limit state reliability index, β_2, reaches the threshold value β_{T2}. Overlay is to replace the cover and the reinforcement of the pier when the ultimate limit state reliability, β_1, reaches the threshold value β_{T1}. Assuming β_{T1} and β_{T2} equal to 3.7 and 1.28 respectively.

6. Case study

An RC bridge pier in [13] is selected and redesigned to illustrate the procedure developed previously. The original diameter of the pier is 2.5 m, and the dead load on the top of the pier is 10000 kN. It is assumed that seismic fortification intensity is 8 degree and the classification of site is class II, and A and T_g is 0.2 g and 0.35 s respectively. To consider the effects of non-uniform corrosion, the pier is divided into submerged zone, splash and tidal zone, and atmospheric zone, the length of each zone is 2.1 m, 6 m, and 5.9 m respectively. Due to the lack of oxygen in the submerged zone, it is assumed that no corrosion will occur, thus, only the corrosion in the other two zones should be considered. The surface chloride concentration at the atmospheric zone and splash and tidal zone are 2.95 and 7.35 kg/m^3 [7] respectively. Note that the moment of the pier is attributed to the horizontal force at the top of the pier which is generated by the earthquake excitation, i.e., the moment increases linearly from top to bottom. The moment at the bottom section is taken as the moment demand in the ultimate limit state function, and the moment at the bottom of atmospheric zone and splash and tidal zone are taken as the moment demand to determine their overlay repair time.

Five types of commonly used materials are used to redesign the bridge pier to compare their cost-effectiveness: normal strength concrete (NSC) with black steel (BS) (named as NSC-BS), NSC with BS with silane soakage on the pier surface (named as NSC-Silane), high performance concrete with 30% fly ash as a replacement of cement with BS (named as HPC-BS), NSC with epoxy coated steel (EC) (named as NSC-EC), and NSC with stainless steel (SS) (named as NSC-SS). The maintenance time for different types of materials are listed in Table 1, and the materials' price (includes the labor cost), and distribution parameters are shown in Table 2, where D is the diameter of the pier, C is the concrete cover depth, A_s is the area of the reinforcement which is expressed by the reinforcement ratio, ρ, and D, s is the stirrup space, d_b and d_{st} are the diameter of longitudinal reinforcement and stirrup respectively, f_y, $f_{y,SS}$, are the yield strength of BS and SS respectively, $f_{c,NSC}$ and $f_{c,HPC}$ are the compressive strength of NSC and HPC respectively, and F is the dead load on the top of the bridge pier. The stirrup encryption area length is 3 m at the bottom of the pier and the stirrup space is 50 mm. Assuming the cap beam is the same for different design materials, thus only the cost of the circular section part of the pier needs to be calculated. The anchorage length requirements for different types of steel are different, and the total length for BS, EC and SS are 13.02, 13.5 and 13.66 m respectively. Due to the higher uncertainty of the yield strength of the corroded reinforcement, the coefficient of variation after corrosion initiation is assumed to be 0.1.

D_c for NSC and HPC is 2*10^{-8} cm^2/s and 0.3595*10^{-8} cm^2/s respectively [14], and Cl_{th} for BS, EC and SS is 1 kg/m^3, 4.6 kg/m^3 and 16.7 kg/m^3 respectively. ρ is in the range of 0.006~0.04.

Table 1 – Maintenance measurements and prices

Measurements	**Execution time**	**Unit**	**Price (￥)**
Routine inspection	All piers, with a time interval of 2 years	m^2	4
Non-destructive evaluation	Starting from the 5th years, with a time interval of 5 years and 10 years for NSC and HPC respectively	m^2	80
Patch-repair	Crack width reaches the limit value, and 10% of the surface area is repaired at a time	m^2	400
Overlay	Flexural capacity reaches the limit value	m^2	1200

Note: Pier using SS only needs routine inspection.

Table 2 – Values and distributions of design variables

Type	Variables	Mean	SD	Distribution
Geometrical design variables	D (m)	1.5~3.0	-	-
	C (mm)	65~100	10	Normal
	A_s (mm^2)	$\pi\rho D^2/4$	-	-
Geometrical variables	s (mm)	100	-	-
	d_b (mm)	32	-	-
	d_{st} (mm)	18	-	-
Strength variables	f_y (MPa)	377.54	25.86	Normal
	$f_{y,SS}$ (MPa)	563.5	38.60	Normal
	$f_{c,NSC}$ (MPa)	43.05	6.72	Normal
	$f_{c,HPC}$ (MPa)	43.05	6.72	Normal
Load variable	F (kN)	10000	-	-

7. Results and conclusions

Based on the approach and related parameters described above, optimization design is conducted using the GlobalSearch solver in Matlab Toolbox (Version 4.0). The optimization results are shown in Table 3, where F_{h} is the horizontal seismic force, Group A is the results which take C_{initial} as the optimization objective, and Group B is the results which take LCC as the optimization objective. The results indicate that the diameters of the pier equal to the lower boundary in both Group A and Group B due to F_{h} reduces with the reduction of the diameter, i.e., the smaller diameter of the pier related to smaller seismic force and small cost. C_{initial} and LCC of NSC-SS in Group A and B, are the same and are the highest among the five types of materials for the high price of SS, although no corrosion happens during the design lifetime due to its high corrosion resistance. The cover in Group B except NSC-SS is bigger than that in Group A, as the maintenance cost is included in the optimization objective. It is obvious that LCC in Group B is lower while C_{initial} is higher than those in Group A.

Table 3 – Optimal design results with $C_{initial}$ and LCC as optimization objectives

Material type		D (cm)	C (mm)	A_s (mm^2)	F_h (kN)	β_1 (t=0)	$C_{initial}$ (￥)	LCC (￥)
$C_{initial}$ as optimization objective (Group A)	NSC-BS	150	65	55493	330.5	4.22	43969	104563
	NSC-Silane	150	65	55493	330.5	4.22	47526	103131
	HPC-BS	150	65	55493	330.5	4.22	46519	68180
	NSC-EC	150	65	55493	330.5	4.22	53489	92609
	NSC-SS	150	81	36995	330.5	4.20	113911	118375
LCC as optimization objective (Group B)	NSC-BS	150	100	57906	330.5	4.21	44860	94217
	NSC-Silane	150	100	57906	330.5	4.21	48416	95867
	HPC-BS	150	69	55493	330.5	4.20	46485	67583
	NSC-EC	150	69	55493	330.5	4.20	53446	92566
	NSC-SS	150	81	36995	330.5	4.20	113911	118375

In this paper, an RBDO procedure is developed for the design of bridge piers, then the procedure is illustrated to the design of an RC bridge pier exposed to seismic and corrosion hazards. The seismic force is calculated using the design acceleration spectrum method. To compare the cost-effectiveness of different materials, five types of commonly used materials are used for the design of the bridge pier. The non-uniform corrosion effect of the pier is considered in the design procedure. The results indicate that increasing the concrete cover is the most effective method to reduce the LCC of coastal bridge piers and the optimization design reduce the LCC of the pier.

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9. Copyrights

10. References

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