- Technical Paper -

EFFECT OF REBAR TYPES ON THE LIFE-CYCLE COST OF RC STRUCTURES IN A MARINE ENVIRONMENT

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ABSTRACT

Application of life-cycle cost (*LCC*) analysis is rapidly increasing that enables optimal decision on rebar type when designing reinforced concrete (RC) structures in a marine environment. In the present paper, it is concentrated on the *LCC* estimation of RC structures taking into consideration the rebar types and uncertainty associated with the prediction of chloride-induced corrosion. In an illustrative example, effect of the difference of marine environment (i.e. region and distance from the coastline) on the *LCC* and time-variant probability associated with the concrete cover cracking is investigated.
Keywords: RC structure, carbon steel reinforcement, stainless steel reinforcement, corrosion, failure probability, life-cycle cost

1. INTRODUCTION

Reinforced concrete (RC) structures with conventional carbon steel (CS) rebars located in a marine environment deteriorates undesirably due to chloride attack. The chloride-induced deterioration causes progressive reduction in safety and reliability of RC structures and infrastructures. Many of RC bridges near coastal area in Japan have suffered the rebar corrosion indicating the serviceability failure [1]. Stewart and Rosowsky [2] reported that steel corrosion in RC bridges can lead to a significant reduction in structural resistance.

Based on the maintenance criteria, RC structures need to be repaired to keep them serviceable [3]. The cost of rehabilitating corroded RC structures now accounts for 50% of total construction expenditure [4]. Weyers [5] reported that the current backlog of bridge maintenance in the United States is equivalent to 28 billion dollars, which mostly attributed to the corrosion of reinforcing steel in concrete. Even by taking special countermeasures at the design stage, the risk of damages due to chloride-induced corrosion of RC structures with CS cannot be completely eliminated [6].

To avoid such kind of strength reduction phenomena and minimize the repair cost during the service life of structures, application of corrosion-resistant stainless steel (SS) rebars of RC structure is gaining momentum. The *LCC* analysis for RC structures can be applied to compare the benefits of using CS and SS [7], and to determine the most economical material [6]. Kilworth and Fallon [8] suggested that expected reduction of maintenance and repair cost could justify the use of costly SS rebars based on the *LCC* analysis.

The corrosion process of steel rebar in RC structures is very complex and it depends on several factors. Structural engineers face continued challenges to accurately assess and model structural life-cycle performance under many kinds of uncertainties. With the presence of uncertainties, it is necessary that long-term structural performance and *LCC* be treated based on reliability concepts and methods. Akiyama et al. [9] presented a computational procedure to integrate the probabilistic hazard associated with airborne chlorides into the life-cycle reliability assessment of RC structures. Val and Stewart [6] performed the *LCC* analysis of RC structures considering the uncertainties associated with the steel corrosion.

In the literature, little attention has been given on the study of effects of the airborne chloride hazard on the probabilistic *LCC*. Moreover, some studies [e.g. 10] have shown that SS rebars may suffer a form of local pitting corrosion which was not considered in previous studies when computing *LCC* of RC structures. In this paper, the effect of rebar types and the marine environment (i.e. region and distance from coastline) on the *LCC* of RC structures is examined taking into consideration the uncertainties associated with the assessment of the airborne chloride hazard and prediction of corrosion process. In an illustrative example, the location suitable to use the SS rebars is identified.

2. HAZARD ASSESSMENT ASSOCIATED WITH AIRBORNE CHLORIDES

To investigate long-term performance of RC structures it is important to perform hazard analysis on coastal marine environment. Due to lack of coastal atmospheric data there is shortage of research in marine

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Table 1 List of considered cities in Japan

NO.	City Name	NO.	City Name	NO.	City Name	NO.	City Name	NO.	City Name
1	Wakkanai	9	Sakada	17	Maizuru	25	Aburatsu	33	Ishigakijima
2	Rumoi	10	Miyako	18	Sakai	26	Uwajima	34	Iriomotejima
3	Otaru	11	Ishinomaki	19	Hamada	27	Muroto	35	Minamidaitojima
4	Esashi	12	Onahama	20	Hagi	28	Shionomisaki	36	Kumejima
5	Tomakomai	13	Niigata	21	Fukuoka	29	Owase	37	Nago
6	Kushiro	14	Fushiki	22	Nagasaki	30	Katsuura	38	Naha
7	Nemuro	15	Wajima	23	Amakusa	31	Yonagunijima		
8	Fukaura	16	Tsuruga	24	Makurazaki	32	Miyakojima		

hazard assessment [11]. Akiyama et al. [9, 12] developed a probabilistic model of airborne chloride hazard considering the spatial-temporal variation for the reliability assessment of RC structures.

To obtain the attenuation relationship between the amount of airborne chlorides and the distance from the coastline, the observed values [13] in Japan have been used. The amount of chloride in the air depends on the several factors including the wind speed, the ratio of sea wind to land wind and distance from coastline. In the horizontal direction the attenuation of C_{air} can be expressed as:

$$C_{air} = 1.29 \cdot r \cdot u^{0.386} \cdot d^{-0.952} \tag{1}$$

where, u is the average wind speed in m/s during the observation period, d is the distance from the coastline in meter, and r is the ratio of sea wind to land wind. Due to the presence of uncertainty, the attenuation is modified as:

$$C_{air} = x_2 \cdot 1.29 \cdot r \cdot (x_1 \cdot u)^{0.386} \cdot d^{-0.952} \tag{2}$$

where, x_1 is the Gaussian random variable associated with the wind speed and x_2 is the lognormal random variable representing the model uncertainty of attenuation. At a specific site, the probability of C_{air} to exceed an assigned value c_{air} is provided by:

$$q_s(c_{air}) = \int_0^{\infty} P\left(x_2 > \frac{c_{air}}{1.29 \cdot r \cdot (x_1 \cdot u)^{0.386} \cdot d^{-0.952}} \right) \times f(x_1)\, dx_1 \tag{3}$$

where, $f(x_1)$ is the probability density function of x_1.

In this study, the hazard assessment is done for 38 cities in Japan. The lists of cities considered are shown in Table 1. Due to space limitation, the hazard curves for only Esashi are depicted in Fig. 1. Fig. 1 shows the curves assuming the distances of 0.1 km and 0.5 km from the coastline.

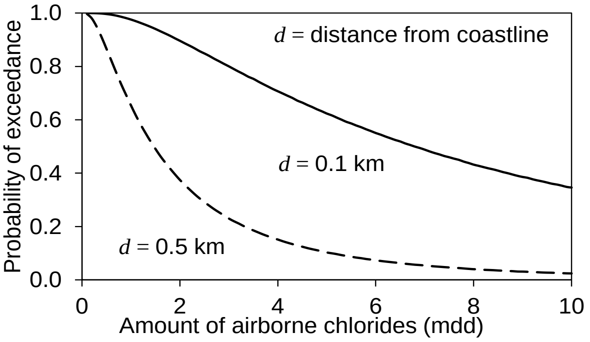


Fig. 1 Hazard curve for amount of airborne chlorides in Esashi

The probability of exceeding a prescribed amount of airborne chloride at Esashi is larger compared with the other cities listed in Table 1. This result implies that the RC structures constructed in Esashi have higher risk for corrosion due to the stronger wind from the Sea of Japan than those at the same distance from the coastline in the other cities. Hazard curves associated with airborne chloride in Fig. 1 are applied to the RC structures for estimating the failure probability.

3. LIFE-CYCLE COST ANALYSIS

For LCC analysis, it is essential to express all attributes and consequences of a decision making in monetary terms. When the benefits of each alternative are the same, the expected $LCC(T)$ up to time T, is provided by:

$$LCC(T) = C_D + C_C + C_{QA} + C_{IN}(T) + C_M(T) + \sum_{i=1}^{M} P_{fi}(T) C_{SFi} \tag{4}$$

$$C_{SF} = 2C_C + C_{QA} \tag{5}$$

where, C_D is the design cost, C_C is the construction cost, C_{QA} is the expected cost of quality assurance, $C_M(T)$ is the expected cost of maintenance, M is the number of independent failure limit states, $C_{IN}(T)$ is the cost of inspection, $P_{fi}(T)$ is the cumulative probability of failure for each limit state and C_{SFi} is the failure cost associated with the occurrence of each limit state.

In Eq. 5 it is assumed that the cost due to cover cracking is twice of construction cost in addition with cost of quality assurance. The reason behind this costing is associated in two phases including the cost for removal (C_C) in the first phase and cost for replacement (C_C+C_{QA}) in the second phase [6].

It is assumed that the cracking of concrete cover is repaired by patching the same thickness of cover with concrete quality as per the original design specification ($C_{QA}=0$). Such repair cannot lead any improvement in performance associated with the durability. To perform present worth analysis of all benefits and costs, the discount rate is assumed to be 2%. In Japan, inspection period for road bridges is five years [14], which is considered in the LCC analysis.

The cost for construction of RC structure is composed of material cost and labor cost. When using CS, the construction cost for RC structures with 30-mm-thick concrete cover and W/C=0.45 is set to be 1.0 as a baseline of construction cost. The replacement of CS by SS reinforcements moderately increases the overall initial costs normally less than 20% [15]. In this study, the construction cost of RC structure with SS reinforcement is taken as 1.05, 1.10, 1.15 and 1.20 C_C, and the effect on *LCC* is investigated later.

Since there is little information on C_D, C_{IN} and C_M in the literature, it is assumed in the *LCC* analysis that C_D, C_{IN} and C_M of RC structures with SS are the same as that of RC structures with CS. They are not needed for the comparative *LCC* analysis.

4. STRUCTURAL PERFORMANCE OF RC STRUCTURE EXPOSED IN AN AIRBORNE CHLORIDE ENVIRONMENT

When RC structure is exposed to chloride attack, the performance function to estimate the probability associated with the occurrence of steel corrosion can be obtained by [11, 12]:

$$g_1 = x_4 C_T - C(c, D_c, C_0, t) < 0 \tag{6}$$

where,

$$C(c, D_c, C_0, t) = x_5 C_0 \times \left\{1 - erf\left(\frac{0.1 \cdot (c + x_6)}{2\sqrt{D_c t}}\right)\right\} \tag{7}$$

$$D_c = x_7 \cdot 10^q \tag{8}$$

$$q = -6.77(W/C)^2 + 10.10(W/C) - 3.14 \tag{9}$$

$$C_0 = x_3 \cdot 0.988 C_{air}^{0.379} \tag{10}$$

C_T is the critical threshold of chloride concentration in kg/m^3, c in mm is the concrete cover specified in design, t is the time after construction in year, W/C is the water to cement ratio, *erf* is the error function, D_c is the coefficient of chloride diffusion in cm^2/year, x_3 is the lognormal random variable representing model uncertainty, x_4 is the Gaussian variable associated with the evaluation of C_T, x_5 is the lognormal variable representing the model uncertainty associated with the estimation of C, x_6 is the normal variable representing the construction error of c, and x_7 is the lognormal variable representing the model uncertainty associated with the estimation of D_c.

Using above equations, the time (t_1) to corrosion initiation can be calculated using the following equation.

$$t_1 = \frac{1}{4D_c}\left\{\frac{0.1(c + x_6)}{erf^{-1}\left(1 - \frac{x_4 C_T}{x_5 C_0}\right)}\right\}^2 \tag{11}$$

When the steel corrosion product (Q_b) becomes larger than the critical threshold (Q_{cr}), corrosion crack occurs. The performance function to estimate the probability of corrosion crack occurrence is provided by [11, 12]:

$$g_2 = x_8 Q_{cr}(c) - Q_b(V, T_{co}, t) < 0 \tag{12}$$

where,

$$Q_{cr}(c) = \eta(W_{c1} + W_{c2}) \tag{13}$$

$$Q_b(V, T_{co}, t) = x_9 \rho_s V_1 (t - t_1) \tag{14}$$

$$W_{c1} = \frac{\rho_s}{\pi(\gamma - 1)} \cdot \alpha_0 \cdot \beta_0 \cdot f_c'^{2/3} \times \left[\frac{0.22\left\{\left(2(c + x_6) + \phi\right)^2 + \phi^2\right\}}{E_c (c + x_6 + \phi)}\right] \tag{15}$$

$$W_{c2} = \alpha_1 \beta_1 \frac{\rho_s}{\pi(\gamma - 1)} \frac{(c + x_6) + \phi}{5(c + x_6) + 3\phi} w_c \tag{16}$$

$$f_c' = -20.5 + 21.0/(W/C) \tag{17}$$

$$\alpha_0 = (-0.0005\phi + 0.028)c + (-0.0292\phi + 1.27) \tag{18}$$

$$\beta_0 = -0.0055 f_c' + 1.07 \tag{19}$$

$$\alpha_1 = (0.0007\phi - 0.04)c + 0.0663\phi + 5.92 \tag{20}$$

$$\beta_1 = -0.0016 f_c' + 1.04 \tag{21}$$

ρ_s is the steel density, γ is the expansion rate of volume of corrosion product, f_c' is the concrete strength in MPa, ϕ is the diameter of steel bar in mm, w_c is the cracking width due to corrosion and 0.1 mm as the threshold of the first crack, V_1 is the corrosion rate of the steel bar before the occurrence of corrosion crack in mm/year, α_0, β_0, α_1 and β_1 are the coefficients, η is the correction factor, x_8 is the lognormal random variable representing the model uncertainty associated with the estimation of Q_{cr}, and x_9 is the lognormal random variable associated with the corrosion rate.

Table 2 Random variables used in *LCC* analysis

Para-meters	Distribution	Mean	COV	Reference
x_1	Normal	1.00	0.105	Meteorolo-
		u = 5.06 m/s		gical Data
x_2	Lognormal	1.06	1.250	[11]
x_3	Lognormal	1.43	1.080	[11]
x_4	Normal	1.00	0.375	[11]
x_5	Lognormal	1.24	0.906	[11]
x_6	Normal	8.50 mm	16.60 mm	[11]
x_7	Lognormal	1.89	1.870	[11]
x_8	Lognormal	1.00	0.352	[11]
x_9	Lognormal	1.00	0.580	[11]

In Table 2, x_1, x_2, x_3, x_4, x_5, x_6, x_7, x_8, and x_9 are random variables associated with wind speed, attenuation of C_{air}, C_0 and C_{air} relation, critical threshold chloride concentration at the occurrence of steel corrosion, estimation by diffusion equation, construction errors of the concrete cover, diffusion coefficient, critical threshold of corrosion amount at crack initiation, and steel corrosion rate, respectively.

Time t_2 after corrosion initiation to corrosion crack occurrence is provided by:

$$t_2 = \frac{x_8 Q_{cr}(c)}{x_9 \rho_s V_1} \tag{22}$$

The time-variant probability $P_f(t)$ associated with cover cracking within time t can be defined as

$$P_f(t) = P_r(t_1 + t_2 \le t) \quad (23)$$

In this research, assuming that t_2 after the steel corrosion initiation is determined independent of the amount of airborne chloride and that t_1 and t_2 are statistically independent, they can be summed up together, as described in Eq. (23).

In this study, Monte Carlo simulation is used to calculate the probability associated with the cover concrete cracking of RC structures with CS and SS rebars. Based on the experimental data in the literature and survey results of existing concrete structures [9, 11, 12], the random variables to estimate $P_f(t)$ are determined as listed in Table 2.

There are different types of SS reinforcement (SS304, SUS316 and SUS410) depending on the chemical composition. In the present study, SUS410 (contained 12% Cr) is considered for comparing the *LCC* of RC structures with CS. The threshold of chloride concentration for the corrosion initiation of CS rebar is 2.03 kg/m^3, whereas that of SS rebar is 9.03 kg/m^3 [16]. Due to low threshold of CS, it is corroded earlier compared with the SS rebars in RC structures. The experimental data on the critical threshold of chloride concentration for SS are still not enough in the literature. In the present study, it is assumed that the mean is 9.03 kg/m^3, and the coefficient of variation (COV) and probabilistic distribution of SS rebar are the same as those of CS rebar.

Also, based on the previous report [6], t_2 of RC structure with SS is assumed to be the same as that with CS. This assumption would provide the overestimation of *LCC* for RC structure with SS [16].

5. *LCC* ESTIMATION

5.1 Probability of cover cracking of RC structures

The probabilities of cover cracking of the RC structures constructed with CS and SS rebars due to corrosion are presented in Fig. 2 and Fig. 3. As shown in Fig. 2 and Fig. 3, the probability of cover cracking of the RC structures near the coastal marine environment is higher and it decreases as the distance from the coastline increases. In case of RC structure with CS located in Esashi at a distance of 0.1 km, which is denoted as SFM_0.1 km, is three times more vulnerable to corrosion than the structure located at 2.0 km from the coastline. The probability of cover cracking is reduced dramatically when SS rebar is used instead of CS in RC structures at all locations. It is observed that the cover cracking probability is decreased by 77 % when SS is used as rebars in RC structure near the coastline.

5.2 Life-cycle cost of RC structures

The *LCC* of the RC structures with CS and SS rebars at the distances from the coastline of 0.1 km, 0.5 km, 1.0 km and 2.0 km are shown in Fig. 4 and Fig. 5. By comparing Fig. 4 and Fig. 5, it is found that the *LCC* of RC structure is larger initially for RC structure with SS. However, because of high corrosion resistance property of SS rebar, it can reduce the number of repair activity for the RC structure with SS. As a result, despite high construction cost at the initial stage, the *LCC* of RC structure with SS becomes lower than that of RC structure with CS near the coastline.

The effect of the distance from the coastline in 38 cities of Japan on the *LCC* of RC structures with CC and CS rebars is examined. The *LCC* for the RC structures located at the distances of 0.1 km, 0.5 km and 2.0 km is shown in Figs. 6(a), (b) and (c), respectively. When the RC structure is located at 0.1 km from the coastline, the *LCC* of RC structure with CS is higher than that with SS if C_C = 1.05. Meanwhile,

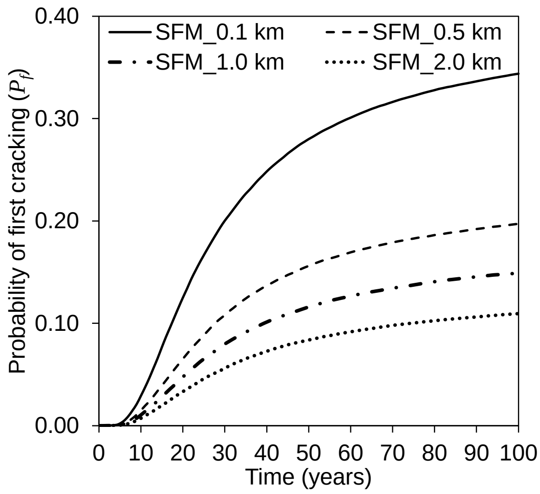


Fig. 2 Probability of cover cracking of RC structure with CS in Esashi

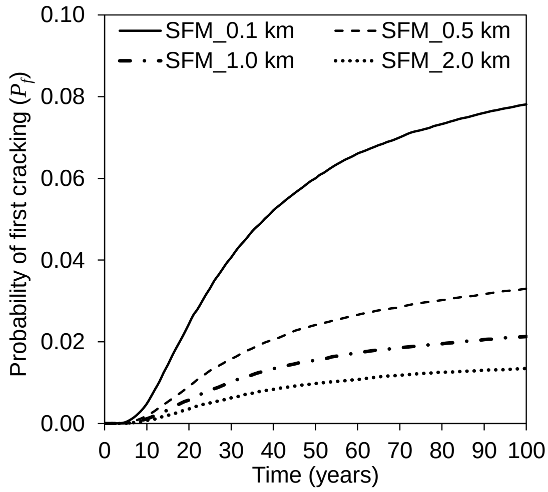


Fig. 3 Probability of cover cracking of RC structure with SS in Esashi

when the distance from the coastline is larger than 0.5 km, the *LCC* for RC structure with SS rebar in 27 cities is higher than that with CS rebar.

The expected repair cost due to corrosion can reduce drastically with the distance from the coastline. The *LCC* of RC structures using CS is reduced in a faster rate than that using SS, and the *LCC* of RC structures gets closer to the initial construction cost. When RC structures are located at 2.0 km from the coastline, the *LCC* of RC structures with CS for all 38 cities is less than that with SS (see Fig. 6c) independent

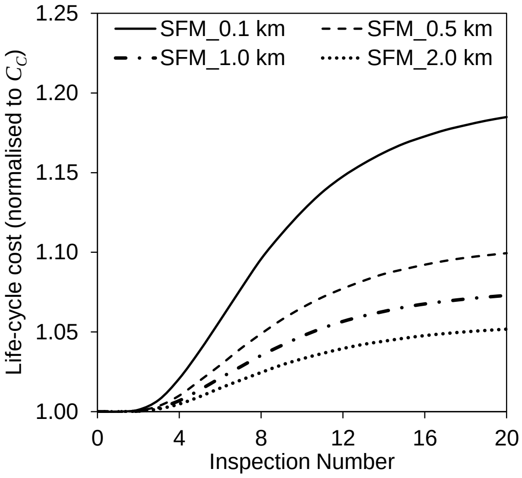


Fig. 4 Life-cycle cost of RC structure with CS in Esashi

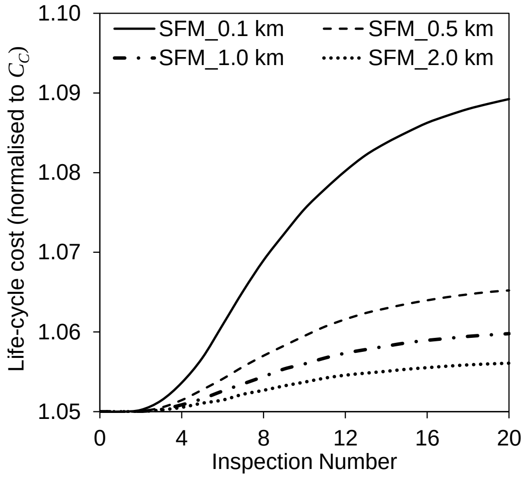
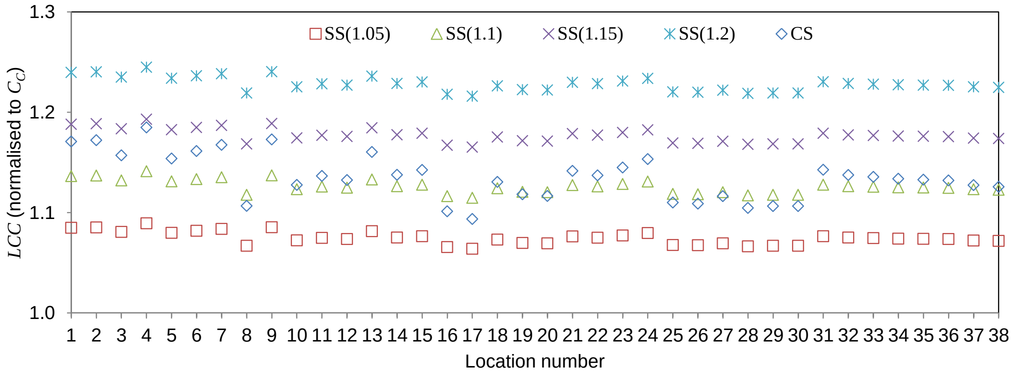
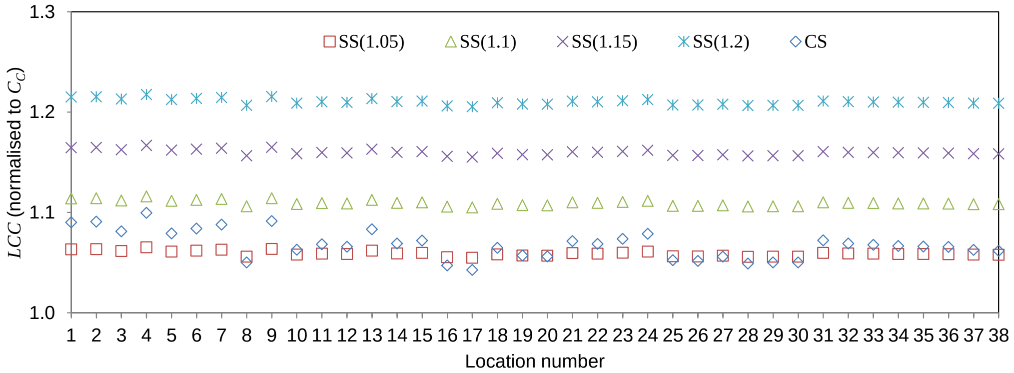


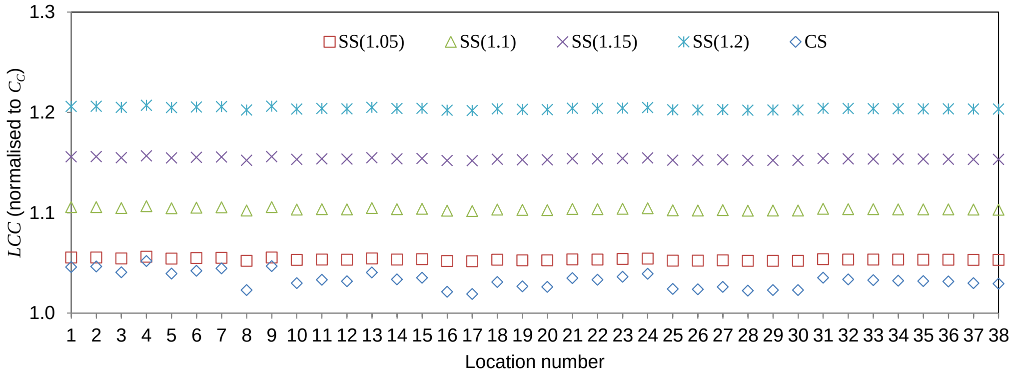
Fig. 5 Life-cycle cost of RC structure with SS in Esashi



(a)



(b)



(c)

Fig. 6 Maximum *LCC* variation of RC structures located at a distance (a) 0.1 km (b) 0.5 km and (c) 2.0 km from coastal marine environment in 38 Cities in Japan of the construction cost of RC structure (i.e. 1.05, 1.10, 1.15 and 1.20 C_C). Use of SS cannot be justified on the *LCC* basis at such the region.

6. CONCLUSIONS

This study presents a procedure for estimating the probabilistic *LCC* of RC structures using CS and SS rebars in a marine environment. Effect of the difference of marine environment on the *LCC* and the probability associated with the concrete cover cracking is investigated in an illustrative example.

From this study, the following conclusions are drawn.

(1) Probabilistic *LCC* analysis was conducted to identify the location suitable to use the SS rebar. The procedure to estimate the *LCC* taking into consideration the uncertainties associated with the airborne chloride hazard and prediction of steel corrosion was presented.

(2) The rate of decrease in *LCC* is higher in case of RC structure with CS than RC structure with SS as the distance from the coastline increases.

(3) It is not justified to design RC structures using high corrosion resistant SS rebar independent of the hazard level associated with the airborne chloride. Within a certain distance from the coastline, RC structures with SS reinforcement can give a cost advantage over those with CS reinforcement.

There is obviously a strong financial incentive to extend the service life of structures. An approach for improving the durability is to use the SS rebars; however, the replacement of the conventional rebars with the SS results in an increase of the initial cost. The *LCC* analysis must be applied for using the high corrosion-resistance materials.

Further research is needed to develop the probabilistic modelling of corrosion associated with the SS reinforcement. Also, *LCC* needs to be estimated based on the other performance indicators (e.g, safety and serviceability).

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