A Study on the Lifecycle Cost of a Concrete Bridge using FRP in Chloride Attack Environments

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ABSTRACT: Analysis was conducted on the life-cycle cost (LCC) of using fiber-reinforced plastic (FRP) to reinforce a concrete bridge exposed to a chloride environment. Japan faces the serious problem of increased maintenance costs from bloc obsolescence of bridges, at a time of shrinking public works budgets. We face a demand for cost reduction and improved durability of infrastructure. FRP is highly durable, but its construction as reinforcement costs more than that for steel. To examine the cost performance of FRP on a concrete bridge (span length: 18 m) exposed to a chloride environment, we compared four cases in terms of LCC and initial construction cost. Case 1 uses conventional concrete cover with protective coating for reinforcement. Case 2 uses FRP only for reinforcement. Case 3 uses FRP reinforcement in conjunction with prestressed concrete (PC) strands. Case 4 uses electrochemical protection. The lowest initial construction cost was for Case 1 (=100%), followed by Case 3 (106%), Case 4 (115%), and Case 2 (123%). The lowest LCC was for Case 3 (69% of the LCC for Case 1), followed by Case 2 (88% of the LCC for Case 1) and Case 4 (130% of the LCC for Case 1). Our study confirms that FRP retrofit of concrete bridges in a chloride attack environment reduces LCC.

1 INTRODUCTION

In Japan, chloride-induced steel corrosion reduces the long-term durability of concrete bridges. Adequate maintenance for existing structures and high durability and long lifespan for new structures are needed. It is imperative to develop new materials and new technologies to meet those demands.

The *Specifications for Highway Bridges* notes that reinforcement against salt damage should be implemented in coastal concrete bridges according to the distance of the bridge from the coast and other geographical characteristics. The cost of conventional reinforcement can be high for bridges exposed to severe conditions; new materials and technology need to be developed.

Fiber reinforced plastic (FRP) has been examined as such a material. Highly durable, light, and strong, FRP is effective in improving the durability of concrete structures and in reducing their maintenance costs. However, the construction cost for concrete bridges reinforced with FRP is higher than for concrete bridges reinforced by conventional methods, because FRP is more expensive than the steel that is used in conventional reinforcement.

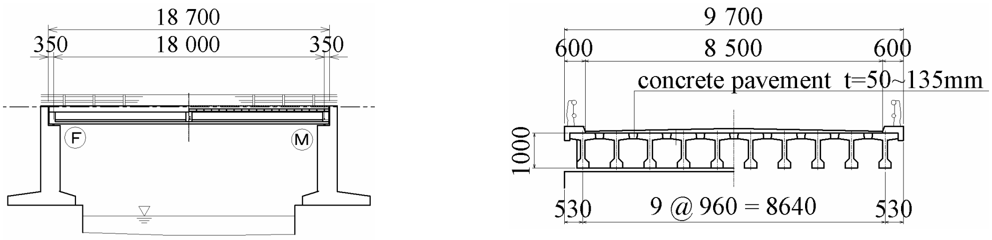
We examined the life-cycle cost (LCC) of a prestressed concrete (PC) bridge whose girders are reinforced by FRP toward examining the LCC performance of FRP as a bridge reinforcement material.

LCC consists of initial construction cost, maintenance cost and removal cost. This study focuses on the first two.

2 OUTLINE OF STUDY

2.1 *Studied bridge*

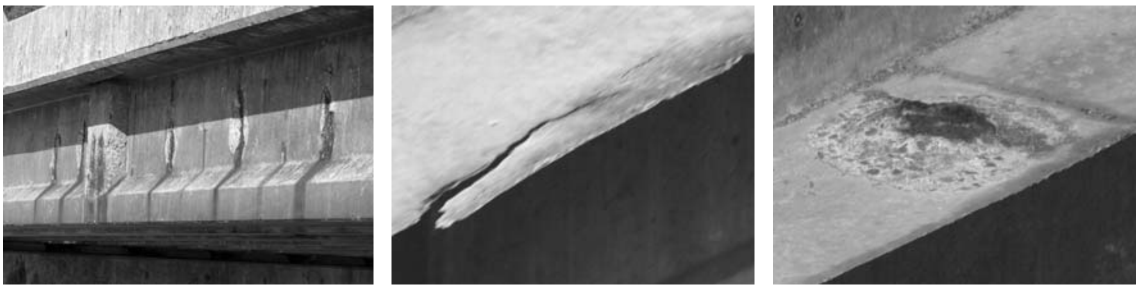
The studied bridge is a PC bridge with a span of 18.0 m and a width of 8.50 m (width of 9.7 m including wheel guards). The design live load is “live load B.”



(A) Side view (B) Cross section

Figure 1. The studied bridge that is a PC T-girder bridge with a span of 18.0 m and a width of 8.50 m.

This PC T-girder bridge was constructed in 1978 (29 years in service at the time of the study). Damage to the bridge includes steel corrosion, and concrete exfoliation and spalling, which are typical of salt damage. The damage is midway between what is described in Japan as the “acceleration stage” and “deterioration stage.



(A) Steel corrosion of stirrup (B) concrete exfoliation (C) concrete spalling

Figure 2. The PC T-girder bridge constructed in 1978 with the damage that is midway between the “acceleration stage” and “deterioration stage”.

2.2 *Performance and classification of measures against chloride-induced deterioration*

A service life of 100 years was assumed, based on the *Specifications for Highway Bridges*. The *Specifications* identify steel corrosion as the determining factor in long-term durability of PC bridges.

In terms of chloride-induced damage, the bridge is classified as a “Class-S bridge,” i.e., one exposed to a severe chloride environment from being constructed within 100 m of the shoreline. The minimum concrete cover over the steel reinforcement is 70 mm. If FRP reinforcement is used, the minimum concrete cover over the FRP is 25 mm, because FRP has high durability.

(A) In a moderate weather with waves (B) In a bad weather with sea water splash

Figure 3. The studied bridge located in a sever chloride attack environment.

2.3 *Material and permissible stress*

The PC girder design requires concrete with a compressive strength of 50 N/mm^2 and a water/cement ratio of 36%.

2.4 *Prediction of deterioration*

(1) Method of predicting chloride ion diffusion
Factors in corrosion of the steel used for girder reinforcement include chloride penetration and concrete carbonation. This study addresses chloride induced damage.

For a concrete girder, deterioration from chloride exposure is predicted by estimating the time before the chloride ion concentration reaches the “corrosion threshold of chloride ion concentration,” i.e., the threshold at which steel bars begin to corrode. Chloride diffusion in this study was predicted using a one-dimensional diffusion equation (Fick’s second law). Here, the chloride ion concentration at the concrete surface is assumed to be constant and on condition that there is no crack.

$$C(x,t) = Co\left(1 - erf\,\frac{x}{2\sqrt{D \cdot t}}\right) + C(x,0) \tag{1}$$

Where C(x, t) is density of chloride ion (kg/m^3) at depth x (cm) and time t (years); Co (kg/m^3) is chloride ion concentration at the surface; D (cm^2/year) is the apparent diffusion coefficient of chloride ion; erf is an error function; C(x, 0) is the initial chloride ion concentration (kg/m^3) of the concrete.
(2) Variables in diffusion equation
The chloride ion concentration at the surface was assumed to be 13.0kg/m^3, based on an onsite investigation and on the reference value in the *Specifications*. The initial chloride ion concentration (kg/m^3) in the concrete was assumed to be 0.3kg/m^3, the upper limit in the *Specifications*.

The diffusion coefficient of chloride ion was calculated, as per the *Specification*, using an equation for calculating the coefficient of normal Portland cement.

$$\log_{10} D = -3.9 \cdot (W/C)^2 + 7.2 \cdot (W/C) - 2.5 \tag{2}$$

Where D (cm^2/year) is the apparent diffusion coefficient, and W/C is the water/cement ratio. It is thought that the corrosion threshold of chloride ion concentration varies with environmental factors and the chloride resistance of the concrete. This study uses 1.2 kg/m^3, a standard value for the corrosion threshold of chloride ion concentration in Japan. The chloride ion concentration calculated in Equation (1) is multiplied by 1.3 to factor in a safety margin in determining concrete cover and water/cement ratio.

2.5 *Estimation of time series change of chloride ion concentration*

The calculation of time series change of chloride ion concentration is shown in Figure 4, in which the horizontal axis indicates depth from the concrete surface and the vertical axis indicates chloride ion concentration. Each curve shows the time series change of chloride ion concentration since construction.

Figure 4 shows that at 100 years after construction, steel rebars 170 mm below the surface of the concrete start to corrode. It is necessary to position steel strands deeper than 170 mm to avoid corrosion when no measures are taken against chloride induced damage.

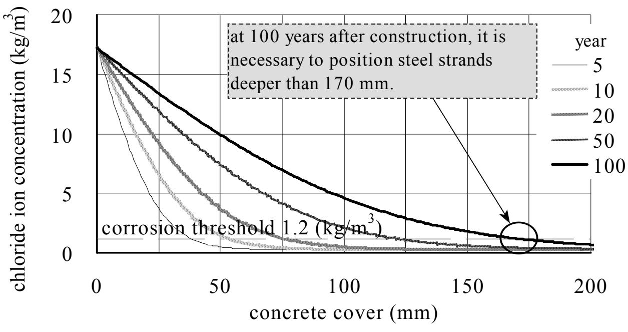


Figure 4. Calculated distribution of chloride ion concentration after different periods.

3 STUDIED CASE

We examined LCC for four cases of reinforcement. Table 1 shows the specifications of the main girders. In all cases, the tensile stress at the lower surface of the main girder was not considered.

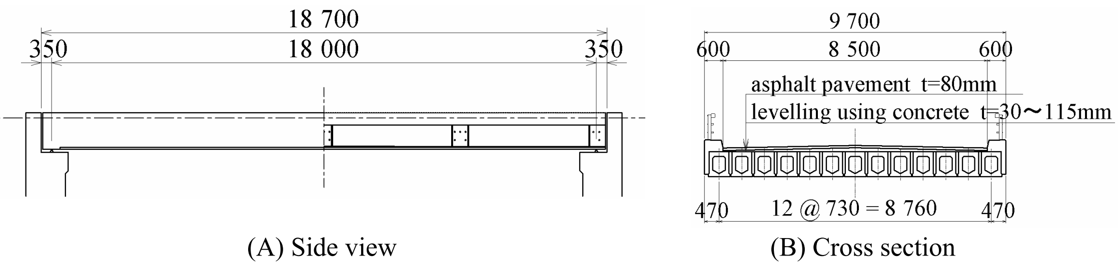
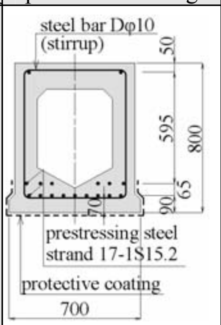
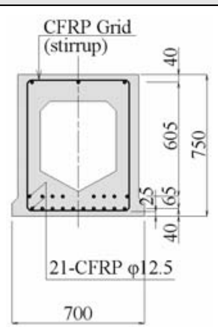
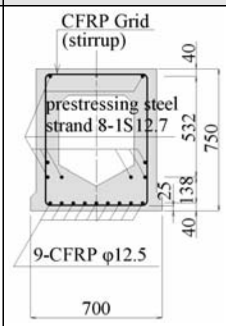
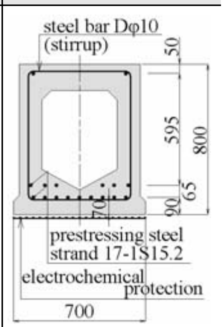


Figure 5. Prestressed concrete hollow bridge for LCC calculation.

(1) Case 1: Conventional reinforcement
This is the standard method of reinforcing against salt damage. The girders (800 mm in height) are reinforced by 17 steel strands (1S15.2) and the girder is given a protective coating. The concrete cover is 70 mm thick.
(2) Case 2: FRP reinforcement
FRP rebars are used as the reinforcement in place of the steel strands used in Case 1. FRP is not deteriorated by salt, and FRP reinforcement eliminates the need to increase the concrete cover thickness or to use a protective coating. In this case, the minimum concrete cover thickness is 25 mm, which affords a lower girder height of 750 mm, i.e., 50 mm lower than Case 1. For the FRP rebars, 21 carbon fiber rebars of CFRPφ12.5 are used.
(3) Case 3: FRP reinforcement and conventional steel strands
To reduce the initial construction cost, FRP is used only for the lowest layer of rebars and stirrups, in combination with conventional reinforcement using steel strands. Steel strands used as inner rebars are positioned 170 mm from the lower surface of the main girder to keep the chloride ion concentration below the corrosion threshold for 100 years in service. The girder height is 750 mm. As rebars, 9 of CFRPφ12.5 and 8 of 1S15.2 are used.
(4) Case 4: Electrochemical protection method
Electrochemical protection method is used at and after construction to mitigate corrosion. The girder height and the number of rebars are the same as in Case 1.

Table 1. Studied cases.

		Case-1	Case-2	Case-3	Case-4
		protective coating	FRP	FRP and steel strand	electrochemical protection
Cross section of girder					
Girder height		800 mm	750 mm	750 mm	800 mm
Tendon	Steel strand	17-SWPR7Bϕ15.2	—	8-SWPR7Bϕ15.2	17-SWPR7Bϕ15.2
	FRP	—	21-CFRPϕ12.5	9-CFRPϕ12.5	—
Stirrup	Steel bar	Dϕ10	—	—	Dϕ10
	FRP	—	CFRP (grid type)	CFRP (grid type)	—

4 LCC CALCULATION

4.1 *Maintenance*

The following maintenance is assumed for each case (Figure 6).

Case 1: Surface repainting and concrete resurfacing in every 16 years; inspection in every 5 years.
Case 2: Inspection in every 10 years.
Case 3: Inspection in every 10 years.
Case 4: Device and Computer program updating in every 25 years; replacement of electrodes in every 50 years. Inspection as necessary.

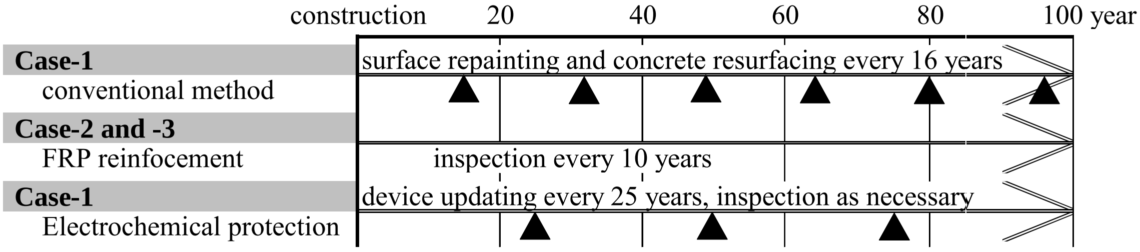


Figure 6. The maintenance of each case.

4.2 *Social discount rate*

In calculating the LCC, the repair cost after a given number of years in service is estimated by converting it to a present value. Called “net present value” (NPV), this figure is expressed by the following equation.

$$LCC = \Sigma\left(\frac{C_t}{(1+r)^t}\right) \tag{3}$$

Where C_t is repair cost after t years, and r is the social discount rate.

The social discount rate is used to convert future value to present value. For an example, a social discount rate of 2% per annum, assuming 0% inflation, means a 2% interest rate on low-risk financial products, such as government bonds. If the social discount rate is high, the repair cost after a given number of years in service is calculated to be small.

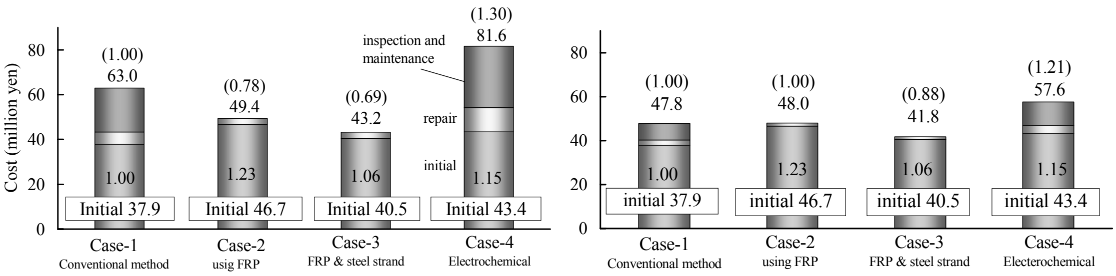
Calculations in this study use social discount rates of 0% and 2% for each case.

4.3 *LCC comparison*

Figure 7 shows LCC calculation results of each case.

When the social discount rate is assumed to be 0%, the smallest LCC is for Case 3: FRP reinforcement used in combination with conventional PC steel. The LCC for Case 3 is 31% less than for Case 1: conventional reinforcement.

The LCC for Case 2, FRP reinforcement, is 21% less than for Case 1. The LCC for Case 4, electrochemical protection method, is the highest; which is 30% more than for Case 1.



(A) Case of social discount rate 0% (B) Case of social discount rate 2%

Figure 7. Result of study on the lifecycle cost.

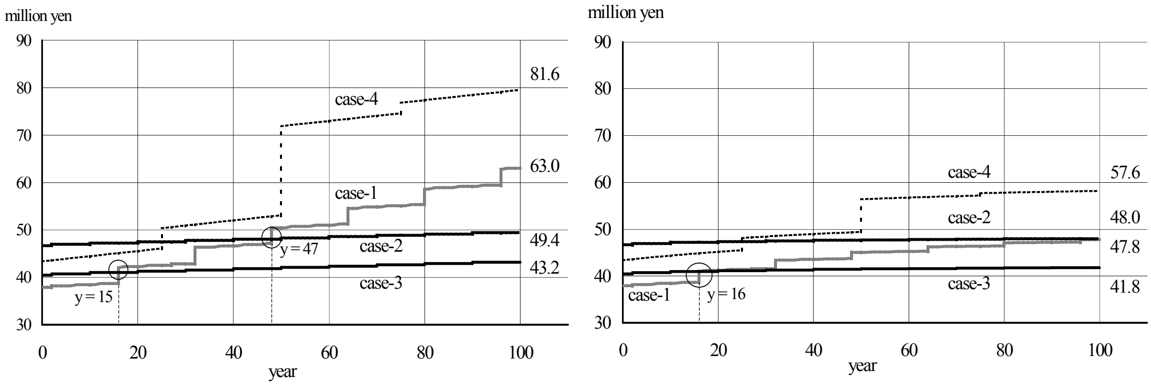
When the social discount rate is assumed to be 2%, the LCC is the same for Case 1 as for Case 2. The difference in LCC between Cases 1 and Case 3 is reduced by 19% (from 31% to 12%) when the social discount rate is assumed to be 0%.

Next, the LCC curves are examined (Figure 8).

When the social discount rate is assumed to be 0%, the LCC for Case 3 becomes smaller than for Case 1 at the 15th year after construction. The LCC for Case 2 becomes smaller than that for Case 1 at the 47th year after construction.

When the social discount rate is assumed to be 2%, the LCC for Case 3 becomes smaller than that for Case 1 at the 16th year after construction.

When the social discount rate is assumed to be 2%, the NPV is smaller than when the social discount rate is 0%. Therefore, maintenance cost in LCC calculation is smaller when the social discount rate is 2% than when the social discount rate is 0%. This is why the LCC for Case 1 remained smaller than that for Case 2 throughout the calculation period.



(A) Case of social discount 0% (B) Case of social discount 2%

Figure 8. Result of study on the lifecycle cost (with passage of year)

5 CONCLUSIONS

The LCC was calculated for four bridge reinforcement methods for securing durability against chloride induced damage during the service life which was assumed to be 100 years, as mandated in the *Specifications for Highway Bridges*.

The study found that preventive measure can reduce the LCC for bridges exposed to severe coastal environmental conditions. FRP reinforcement is found to be effective in improving the durability of concrete bridges, because FRP has high performance for resistance to deterioration.

The study also found that the following issues need to be addressed for future studies.

1) Reliability of long-term durability of FRP.
2) Long-term monitoring to collect and examine data of bridges in service.
3) Reduction of initial construction cost with use of FRP reinforcement while maintaining performance of bridges.
4) Establishment of an LCC calculation method for long-term maintenance plans of bridges and other concrete structures.

The authors hope this study will help in developing new reinforcement methods using FRP for concrete bridges exposed to severe environments.

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