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Research Article

Life-Cycle Management Strategy on Steel Girders in Bridges

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The major problems affecting the service life of bridges are related to various factors such as fatigue-sensitive details, increased service loads, corrosion deterioration, and the lack of proper maintenance. Among them, corrosion deterioration and fatigue damages of structures particularly to steel girder bridges are the most common ones. Bridges of different structural forms, at different locations or under different climates, may suffer from various degrees of deterioration. Steel girders at different positions of a bridge may also suffer from different degrees of damage. How to effectively maintain the bridge asset at a minimal cost and how to predict the time for future works are crucial, particularly when government funding sources become stretched. A comprehensive bridge management framework assisting stakeholders to appropriately and reasonably prioritize their future maintenance-related works in their bridge stocks, such that stakeholders can better allocate the limited resources, is utmost concerned. This paper proposes an integrated life-cycle management (LCM) strategy on steel girders in bridges in which corrosion deterioration and fatigue damage prediction models are mapped with girders' performance conditions. A practical example to demonstrate the applicability of the proposed LCM strategy is also illustrated.

1. Introduction

In recent decades, the long life of active use of transport infrastructure has become a worldwide issue. The developmental spending or investment for preparation of new infrastructures is becoming very difficult in all over the world particularly due to the poor economic condition. The life extension of the bridges not just makes the large economic profit, but also alleviates the financial burden on asset management, and effective decreases in the global warming and other environmental pollutions in life cycle of structure. Bridges are essential in transport infrastructure, and bridge maintenance or replacement is one of the largest expenditure items in the bridge life span.

In Japan, much infrastructure was constructed to support the spreading transportation network from 1950's to the 1970's. Recently, many infrastructures in Japan are getting old, and the numbers of bridges that have been in active service more than 50 years are increasing dramatically, and it is expected that these bridges will be more than 50,000 in 2021. From reasons, the inspection, maintenance, and rehabilitation planning are very important problem for long-lived active use of bridge and infrastructure [1]. In Australia, there are over 30,000 roads and rail bridges. For example, the Queensland government allocated $350 million towards replacing approximately 100 old and obsolete road bridges in regional Queensland over the next five years, from 2006 to 2010 [2]. In USA, more than 43% of the bridges are made of steel. Currently, there are 190,000 steel bridges (simply supported and continuous) of which over 40,000 (25%) are structurally deficient and over 35,000 (18.5%) are functionally obsolete [3].

2. Steel Bridge Problems

The major problems affecting the service life of bridges are related to various factors such as fatigue-sensitive details, increased service loads, corrosion deterioration, and the lack of proper maintenance [4]. Among typical types of damage as shown in Figure 1, corrosion deterioration and fatigue

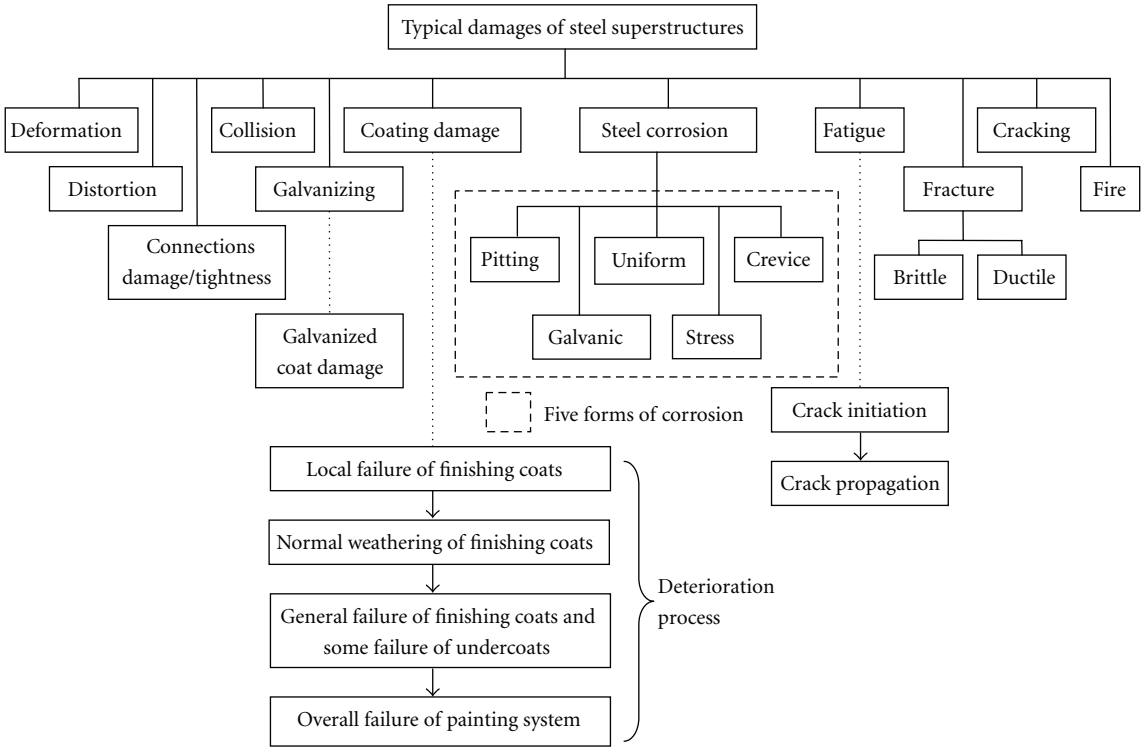

Figure 1: Typical types of damage of steel superstructures.

damages of structures particularly to steel girder bridges are the most common ones which are influenced by the environment and the vehicle loadings and its stress ranges.

In conventional practice, inspection, maintenance, or repair works are scheduled in accordance with the maintenance and rehabilitation manual and the maintenance authority would conduct scheduled works and record all the findings accordingly. However, the manual basically is a typical one rather than being specially made for a particular asset. Bridges of different structural forms, at different locations or under different climates, may suffer from various degrees of deterioration. Steel girders at different positions of a bridge may also suffer from different degrees of damage. How to effectively maintain the bridge asset at a minimal cost and how to predict the time for future works are important issues, when government funding sources become stretched.

To relieve the long-term financial burden on asset maintenance, whole life-cycle management (LCM) concept has been introduced in recent decade and is now getting more important in engineering design, construction, and management. In the whole LCM concept, the prediction of structural member deterioration is essential to plan for future maintenance actions. This paper proposes an integrated LCM strategy on steel girders in bridges in which corrosion deterioration and fatigue damage prediction models are mapped with girders' performance conditions. A practical example to demonstrate the applicability of the proposed strategy is also illustrated.

3. LCM in Steel Girder Bridge

This paper proposes an integrated LCM strategy considering three structural assessment factors altogether simultaneously: (1) the serviceability limit: deflection; (2) the ultimate limits: moment and shear, and (3) the fatigue strength limit. Clearly, the ultimate limits are not only two considerations, moment and shear. The bearing capacity can also be affected by corrosion. However, in common practice, if a steel bridge will not receive proper maintenance and painting, it will probably be constructed with bearing stiffeners to increase the bearing capacity. In order to reduce the complexity in showing the implementation of proposed integrated LCM strategy, the technical analysis of bearing behavior is not included in the paper. A steel girder bridge is provided as an example to demonstrate the use of the proposed strategy with corrosion deterioration and fatigue damage models being incorporated.

4. Corrosion Deterioration Model

Except for high-performance steel such as anticorrosion weathering steel, steel girder bridges are usually subject to corrosion to certain degrees. If undetected over a period of time, corrosion will weaken webs and flanges of steel girders by reducing the material thickness and possibly lead to dangerous structural failures [7].

In serviceability limit analysis, measurements of remaining thickness of corroded steel web and bottom flange are

Table 1: Statistical parameters for A and B [5, 6].

Parameters	Carbon steel		Weathering steel	
	A (μm)	B	A (μm)	B
(a) Rural environment				
Mean value, μ	34.0	0.65	33.3	0.498
Coefficient of variation, σ/μ	0.09	0.10	0.34	0.09
Coefficient of correlation, ρ_{AB}	—	—	−0.05	—
(b) Urban Environment				
Mean value, μ	80.2	0.593	50.7	0.567
Coefficient of variation, σ/μ	0.42	0.4	0.30	0.37
Coefficient of correlation, ρ_{AB}	0.68	—	0.19	—
(c) Marine Environment				
Mean value, μ	70.6	0.789	40.2	0.557
Coefficient of variation, σ/μ	0.66	0.49	0.22	0.10
Coefficient of correlation, ρ_{AB}	−0.31	—	−0.45	—

commonly considered. The effective thicknesses of webs and flanges are reduced with time as [7]:

$$t_f(t) = t_{f0} - C(t), \tag{1a}$$

$$t_w(t) = t_{w0} - 2C(t), \tag{1b}$$

where t_{f0} = the initial flange thickness (mm), t_{w0} = the initial web thickness (mm), $C(t)$ = the average corrosion penetration (mm) at time t.

Corrosion is influenced by the environment such as the amount of moisture in the air and the presence of salt. There is a common agreement that the corrosion time versus penetration rate can be modeled, with a good approximation, by an exponential function [8]:

$$C(t) = At^B, \tag{2}$$

where $C(t)$ = average corrosion penetration in micrometres (μm) after t years, t = time (years) of exposure, A = corrosion loss parameter after one year of exposure, and B = parameter determined from regression analysis of experimental data.

Parameters A and B were determined by Albrecht and Naeemi [5] and further verified by Kayser [6] as shown in Table 1.

Researchers have pursued extensive studies to predict time-variant corrosion propagation to capture the actual corrosion. However, these studies often neglect the influence of the periodic repainting effect on the corrosion process [3, 6, 8–10].

Lee et al. [7], based on previous studies, introduced a modified corrosion propagation model with periodic repainting as shown in (3). Lee's corrosion model is adopted for service life prediction of steel girders in this paper.

$$p_i(t) = \begin{cases} C(t - iT_{\mathrm{REP}} - T_{\mathrm{CI}})^m & \text{for } (i)T_{\mathrm{REP}} + \mathrm{T}_{\mathrm{CI}} \\ & \leq t < (i+1)T_{\mathrm{REP}} \\ p_{i-1}(iT_{\mathrm{REP}}), & \text{otherwise,} \end{cases} \tag{3}$$

where $p_i(t)$ is corrosion propagation depth in micrometer (μm) at time t in years during ith repainting period; C is random corrosion rate parameter; m is random time-order parameter; and T_{CI}, T_{REP} = random corrosion initiation and periodic repainting period (yrs), respectively.

5. Fatigue Damage Model

Several models have been developed to describe the process of fatigue damage, including the S-N model, the Miner's linear cumulative fatigue damage model, and the crack growth model under linear-elastic fracture mechanics (LEFM) approach [8]. S-N and Miner's models both have limitation in addressing the probabilistic nature, whereas LEFM approach based on crack propagation theory yields more accurate results for fatigue and fracture reliability assessment if the current crack size is measured [5]. Since the effect of crack size is taken into consideration, this approach yields more accurate results for fatigue and fracture reliability assessment if the current crack size can be measured [6]. In welded bridge details, the welding process inherently results in initial flaws from which crack growth may occur under cyclic loadings [9].

The commonly used crack growth model is the Paris-Erdogan model which is simplified to determine the required cycles for fatigue failure and this model is adopted for service life prediction in this paper. The number of cycle required to grow for fatigue crack can be estimated by taking integration from initial crack dimension a_0 to critical crack dimension a_c [11]. Zhao's model equation is shown in (4):

$$\int_{a_0}^{a_c} \frac{da}{[F(a)\sqrt{\pi a}]^m} = \int_{N_0}^{N_c} CS^m dN, \tag{4}$$

where $F(a) = F_e F_s F_w F_g$ is defined as the crack-size-dependent correction factor; F_e is crack the shape correction factor; F_s is the front free surface correction factor; F_w is the finite plate width correction factor; F_g is the stress gradient

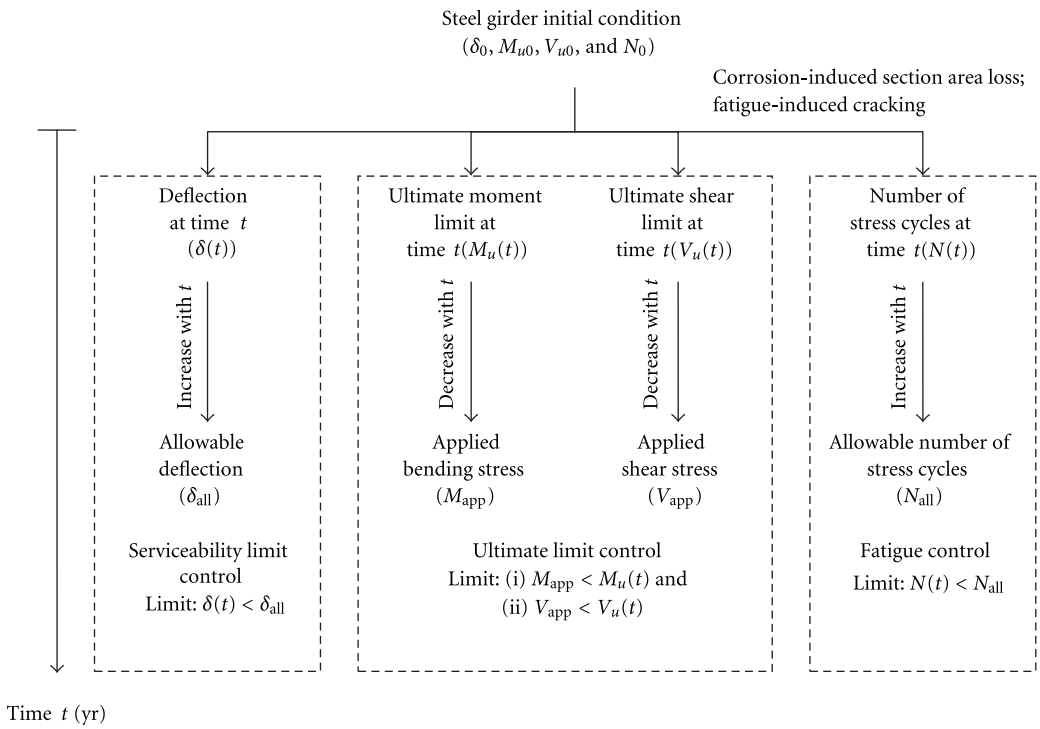

Figure 2: Integrated LCM strategy on steel bridge girders due to corrosion and fatigue.

correction factor; S is the stress range; C is the crack growth constant:

$$\int_{N_0}^{N_c} CS^m dN = C\sum_{i=1}^{k} S_i^m (N_{ic} - N_{i0})$$

$$\int_{N_0}^{N_c} CS^m dN = C\sum_{i=1}^{k} S_i^m (N_{ic} - N_{i0}) \tag{5}$$

$$= CS_{r\text{Miner}}^m N_T.$$

Since randomly variable loading was involved in every case of fatigue crack propagation, an effective stress intensity range was used based on Miner's rule and the corresponding Miner's effective stress. $S_{r\text{Miner}}$ is the effective stress range while the N_T is the required number of cycles to cause fatigue failure. The crack growth component m equal to 3 has been observed to be applicable to basic crack growth rate data for structural steels as well as test data on welded members. The corresponding mean value of growth constant C was found to be 1.26×10^{-13} [11, 12]. Equation (4) can be further simplified to determine the required cycles for fatigue failure:

$$N_T = \int_{a_0}^{a_c} \frac{da}{1.26 \times 10^{-13} [F(a) S_{r\text{Miner}} \sqrt{\pi a}]^3}. \tag{6}$$

Having estimated the fatigue life in cycles, N_T, for the steel component, the data on average daily traffic specific to the bridge roadway, distribution of traffic volume per traffic lane, and expected traffic growth in future years are used to convert N_T in terms of time measures in years. If t = fatigue life in years, then

$$N_T = (1 + r)^t \text{ADTT}(365) S_c, \tag{7}$$

where r is the yearly rate of traffic volume increase, ADTT is a random variable representing the average daily truck traffic and S_c is a random variable representing equivalent number of stress range cycles per truck crossing. The remaining fatigue life will be $t - t_0$, where t_0 is the current age of the steel component in years [13].

6. Integrated Life-Cycle Management Strategy

Corrosion deterioration and fatigue damages may weaken structural members and result in the increment of deflection, the reduction of ultimate bending, and shear strengths and the reduction of fatigue strength. The above corrosion and fatigue models with integration of predefined limits as below form service life prediction models to predict the service condition of steel members at any time throughout its life.

Conventional LCM is based on fixed predefined limit states either the serviceability limit (i.e., deflection) or the ultimate limits (i.e., moment and shear capacities) or the fatigue strength limit. However, it may not be always true that the one limit dominates over one another or vice versa in all circumstances. This paper proposes collective considerations of the serviceability limit:deflection, the ultimate limits:moment and shear, and the fatigue strength limit simultaneously which are essential in the further life-cycle cost (LCC) analysis. This proposed LCM strategy on steel girders covering serviceability limit, ultimate limits, and fatigue strength limit is graphically illustrated in Figure 2.

7. Definition of Limits

All structures have two basic requirements in common: safety from collapse and satisfactory performance of the structure for its intended use. The limit states usually define the various ways in which a structure fails to satisfy these basic requirements.

7.1. Serviceability Limit. Serviceability limit states usually relate to satisfactory performance and correspond to excessive deflection, vibration and local deformation. In this paper, the serviceability limit is defined in terms of deflection of a girder at any time, $\delta(t)$, in the design life. If the deflection exceeds the code requirement, δ_{all}, a serviceability failure is assumed to occur. It is also assumed that replacement or rehabilitation works must be conducted before girders reaching its deflection limit.

7.2. Ultimate Limits. Ultimate limit states usually relate to safety and correspond to strength, stability, and very large deformation. In the paper, the ultimate limits are governed by the structural capacity condition in bending, M, and shear, V, for any time in the life span. It is assumed that replacement or rehabilitation works must be conducted before the ultimate moment or shear capacity of the girder, $M_u(t)$ and $V_u(t)$, reaches the total applied moment or shear.

7.3. Fatigue Strength Limit. The fatigue strength limit suggested in this paper is defined in terms of accumulated numbers of stress cycles at any time, $N(t)$, throughout its life. If the number of stress cycles exceeds to its allowable, N_{all}, a fatigue failure is assumed to occur. Similarly, it is assumed that replacement or rehabilitation works must be conducted before reaching the limit.

7.4. Performance-Based Models. To mathematically present the condition of structural members towards its limits at particular time, buffer functions, $F(t)$, measured in percentagefor each limit are then formulated accordingly as (8) to (11) where subscripts δ, M, V, and N for $F(t)$ refer to buffer functions for deflection, moment, shear and fatigue strength, respectively, and subscripts DL, SDL, LL, and I for M or V refer to moment or shear induced by dead load, superimposed dead load, live load, and impact load respectively:

$$F_\delta(t) = \frac{\delta_{\text{all}} - \delta(t)}{\delta_{\text{all}}}, \tag{8}$$

$$F_M(t) = \frac{M_{\text{u}}(t) - M_{\text{DL}} - M_{\text{SDL}} - M_{\text{LL}} - M_{\text{I}}}{M_{\text{u}}(t)}, \tag{9}$$

$$F_V(t) = \frac{V_{\text{u}}(t) - V_{\text{DL}} - V_{\text{SDL}} - V_{\text{LL}} - V_{\text{I}}}{V_{\text{u}}(t)}, \tag{10}$$

$$F_N(t) = \frac{N_{\text{all}} - N(t)}{N_{\text{all}}}. \tag{11}$$

8. Assignment of Condition States to the Steel Girder

In the proposed LCM strategy, four performance condition states, Good (G), Satisfactory (S), Fair (F), and Poor (P) are assigned to the percentage range of the deflection, the moment, the shear, and the fatigue strength buffer as mentioned above, respectively. Minimum acceptable criteria for Good ($G_{\min}$), Satisfactory ($S_{\min}$), and Fair ($F_{\min}$) conditions are also needed to be defined.

$$\Omega_j = \begin{cases} G & F_\delta(t) \geq G_{\min}, \\ S & S_{\min} \leq F_\delta(t) < G_{\min}, \\ F & F_{\min} \leq F_\delta(t) < S_{\min}, \\ P & F_\delta(t) < F_{\min}. \end{cases} \tag{12}$$

Let Ω_j denote the set of possible limit states, then, $j = 1, 2, 3$, and 4 with respect to different limits: deflection, bending moment, shear, and fatigue accordingly. The general performance condition model as (12) is further modified as follows.

Serviceability Limit. For deflection:

$$\Omega_1 = \begin{cases} G & F_\delta(t) \geq 15\%; \\ S & 10\% \leq F_\delta(t) < 15\%; \\ F & 5\% \leq F_\delta(t) < 10\%; \\ P & F_\delta(t) < 5\%. \end{cases} \tag{13}$$

Ultimate Limits. For bending moment:

$$\Omega_2 = \begin{cases} G & F_M(t) \geq 40\%, \\ S & 30\% \leq F_M(t) < 40\%, \\ F & 20\% \leq F_M(t) < 30\%, \\ P & F_M(t) < 20\%, \end{cases} \tag{14}$$

for shear:

$$\Omega_3 = \begin{cases} G & F_V(t) \geq 40\%, \\ S & 30\% \leq F_V(t) < 40\%, \\ F & 20\% \leq F_V(t) < 30\%, \\ P & F_V(t) < 20\%. \end{cases} \tag{15}$$

Fatigue Strength Limit. For fatigue strength:

$$\Omega_4 = \begin{cases} G & F_N(t) \geq 40\%, \\ S & 30\% \leq F_N(t) < 40\%, \\ F & 20\% \leq F_N(t) < 30\%, \\ P & F_N(t) < 20\%. \end{cases} \tag{16}$$

The percentage ranges of buffers for different limit states are in fact various dependent on the stakeholders' decisions. Several factors, such as long-term costs, project risks, environmental policies, or local maintenance practices, may affect the decision of stakeholders. In this paper, for demonstration purpose, the percentage ranges of buffers are

assumed as shown in (13) to (16) and "F" condition state is also assumed as the minimum acceptable condition level. Once acceptable service condition states for each limit have been predefined, future replacement or rehabilitation works shall be carried out at the time before reaching its acceptable condition limits. The predicted action time will then be adopted in the life-cycle cost model for the subsequent cost-benefit analysis.

8.1. Options for LCM Strategy. In the proposed LCM strategy, the service life limits are generalized as three options.

Option 1: Serviceability Limits Control. Deflection:

$$t_{\text{lim}} = F_{\delta}(t) \ < 5\%. \tag{17}$$

Option 2: Ultimate Limits Control. Shear and moment capacities at time t:

$$t_{\text{lim}} = F_V(t) \text{or } F_M(t) \leq 20\%. \tag{18}$$

Option 3: Fatigue strength Limit Control. Fatigue strength at time t:

$$t_{\text{lim}} = \ F_N(t) \leq 20\%. \tag{19}$$

9. Methodology of the Integrated Life-Cycle Cost (LCC) Model

LCC model is commonly used as the basis to evaluate the cost effectiveness of different management strategies. In the model, proper actions taken at appropriate time are a crucial element for the model accuracy. The LCC of a structure is a combination of the present values of all future costs of occurring within the life span of the structure. The minimum expected LCC denoted as LCC_{ET} has been the most widely used criterion in design optimization of new structural systems considering lifetime performance. The mathematical presentation of the expected LCC of a structure over T years of life span can be generalized as:

$$\text{LCC}_{\text{ET}} = \sum_{i=1}^{n} \sum_{t=0}^{T} \left(\beta_i(t) \frac{C_i(t)}{(1+r)^t} \right), \tag{20}$$

where LCC_{ET} is expected total life-cycle cost, $C_i(t)$ is cost of future action i at time t, β_i is probability of occurrence of future action i, r is a discount rate (constant discount rate assumed in the paper), i is type of future actions, t is time in year, and T is service life span in years. The cost of future actions can be further subdivided into expected design cost $C_{\text{des}}(t)$, expected construction cost $C_{\text{con}}(t)$, expected inspection cost $C_{\text{ins}}(t)$, expected maintenance cost $C_m(t)$, expected replacement and rehabilitation cost $C_{\text{rep}}(t)$, expected demolition cost $C_{\text{dem}}(t)$, and expected failure cost $C_{\text{fai}}(t)$, respectively. To assist better understanding of possible strategies-induced costs, the scopes of inspection, maintenance, and replacement and rehabilitation works are defined and elaborated.

10. Inspection, Maintenance, and Replacement and Rehabilitation Strategies

10.1. Inspection Strategy. Inspection strategies vary, depending on the policy of the stakeholders. Frequent inspections provide more updated information for future maintenance plans. However, the LCC of inspection work is usually insignificant in comparison to the whole LCC of the structure, and a cost-effective management approach should be maintained [17]. The cost of inspection work (C_{ins}) is generally categorized into two kinds with defined scopes as follows.

(1) General Inspection ($C_{\text{ins},1}$) A visual inspection can be conducted annually for obvious defects which might lead to safety problems or lead to loss of use of the structure or restriction of use [17]. Visual inspections allow discovering the rusting or loss of material sections. Steel members should be examined regarding (i) the condition of its protective system including protective painting or galvanizing; (ii) the condition of materials towards corrosion; (iii) the condition of connections; (iv) cracking or fracture defects; and (v) structural deformation or distortion.

(2) Detailed Inspection ($C_{\text{ins},2}$) Inspection work includes visual examination of all visible and accessible parts of a structure. Some minor nondestructive inspection (NDI) of representative areas would be carried out. NDI methods commonly include radiography and ultrasonics for the determination of internal defects while dye penetration and magnetic particle inspection for the detection of surface defects. The purpose of detailed inspection is to verify the deterioration state of the structure, if it is on the schedule R&R plan. Thus, the detailed inspection time is adjusted based on simulated R&R time. It is assumed that detailed inspection works shall be conducted before replacement or rehabilitation works taken place [18].

10.2. Maintenance Strategy. Maintenance of protective coatings is important for visual and physical preservation of steel components. A durable paint combination with long maintenance intervals is not only an economical but also an environmentally acceptable solution [18]. Typically, there are three types of coatings for corrosion rate reduction which are paints, steel galvanizing, and oxidized steel formed on weathering steel of a combination of these coatings. Because of the difficulty in galvanizing large sections of steel elements, the industry has generally used paints for protection. Paints or high-performance coatings, as some of the newer systems are known, are separated into three categories: the inhibitive primer, the sacrificial primer, and the barrier coat [19]. Once defects on coating surface have been found, proper cleaning on the surface and subsequent repainting works should be taken place.

10.3. Replacement and Rehabilitation (R&R) Strategy. Repairing process of steel structural elements usually in many cases is very complex and it requires the use of advanced material solution and techniques. The most important damage to the steel bridge elements can be classified into

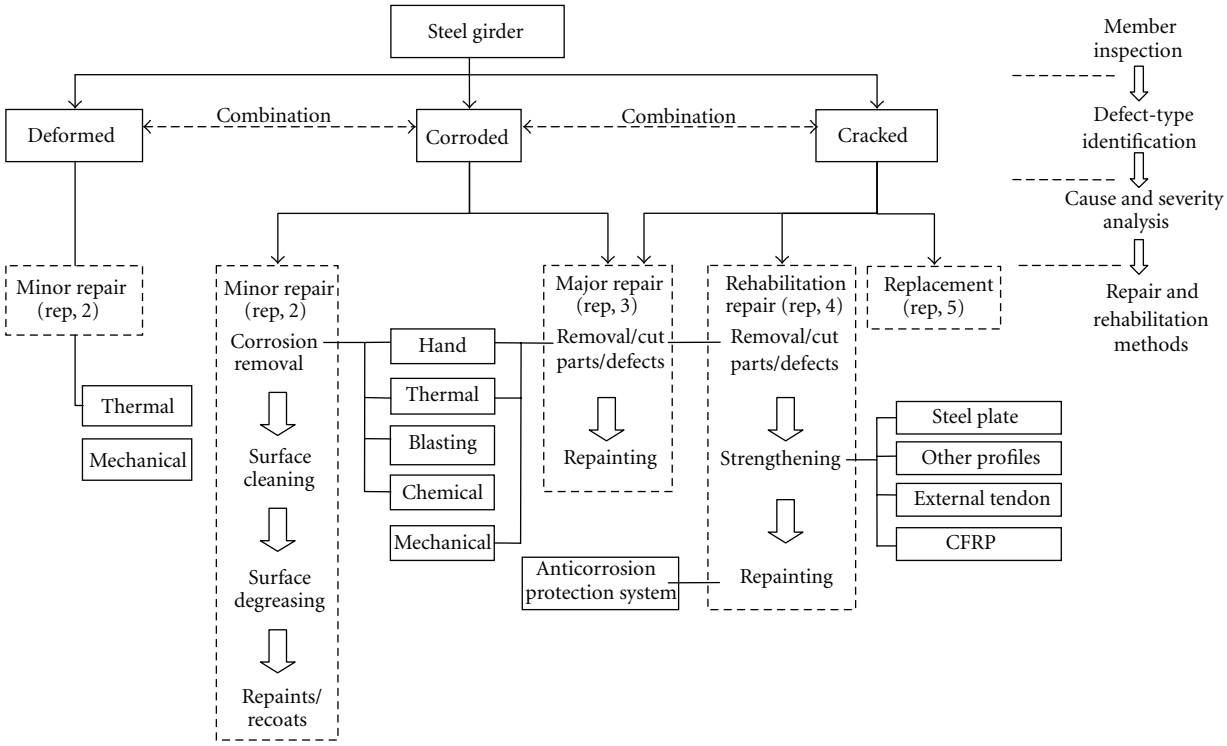

Figure 3: Replacement and rehabilitation methods for conventional steel girder.

Table 2: Proposed replacement and rehabilitation action set.

Action description	Action matrix	Possible replacement and rehabilitation methods
Do nothing	Rep, 1	Observation
Minor repair	Rep, 2	Corrosion removal, surface cleaning, degreasing, and repainting Rectify deformed parts
Major repair	Rep, 3	Removal/cut defects and parts and repainting
Rehabilitation repair	Rep, 4	Removal/cut defects and parts, strengthening, and repainting Structure strengthening by steel plate Structure strengthening by CFRP Structure strengthening by external tendons
Replacement	Rep, 5	Structure replaced by new members

three groups: (i) corrosion destruction of the members and/or their joints; (ii) fatigue effects in steel and its brittle fracture; (iii) mechanical fracture, including collision, of the elements themselves and/or the joints.

Before carrying out any rehabilitation works, the cause of defects and the severity of such defects towards the main structural system should be identified and subsequently analyzed. Depending on the severity and the type of damage causes, different rehabilitation techniques can be applied. General classification of the typical R&R methods for steel girder is proposed and illustrated in Figure 3. Any R&R work should be considered individually and it should be preceded by evaluation and assessment of the condition of the structure, the relevant theoretical analysis as well as the selection of an appropriate repair technology [20].

Regarding the degree of R&R works, it may be classified into five different actions as shown in Table 2 and defined with specified meanings and scopes.

(1) Do nothing (Rep, 1): it means no action is carried out and there is no change in the condition of the structure.

(2) Minor repair (Rep, 2): it provides no improvement in durability performance but slows the deterioration rate such that the condition of the structure or its components could be maintained for a certain further period.

(3) Major repair (Rep, 3): it provides no improvement in durability performance but restores the durability, structural strength, and function or appearance of the

Table 3: Moments and shears due to dead and superimposed dead loads (M_{DL}, M_{SDL}, V_{DL}, and V_{SDL}).

Girder nos.	Position	Due to dead load		Due to superimposed dead	
		Mean M_{DL} (kNm)	Mean V_{DL} (kN)	Mean M_{SDL} (kNm)	Mean V_{SDL} (kN)
1 and 5	Mid span	1,185.30	—	612.20	—
	End of cover plate	314.47	—	162.42	—
	Support	—	169.33	—	87.46
2, 3, and 4	Mid span	1,185.30	—	214.67	—
	End of cover plate	314.47	—	56.95	—
	Support	—	169.33	—	30.67

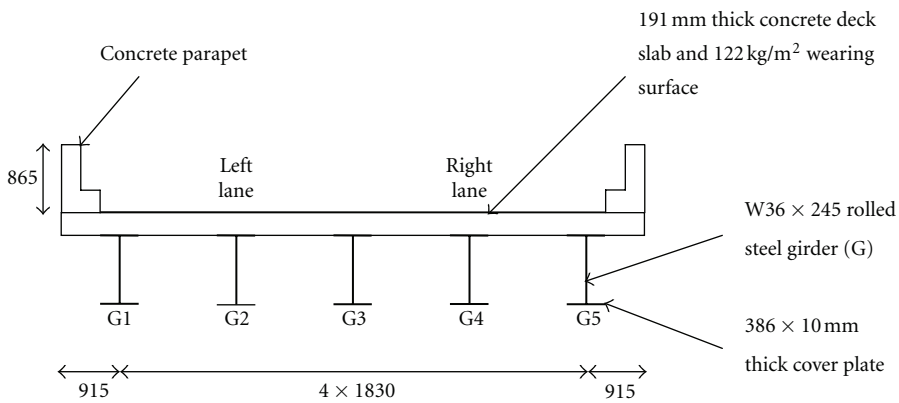

Figure 4: Typical cross section of two-lane rolled steel girder bridge (unit: mm).

structure. The condition of the structure or its components is improved. However, similar damage may reoccur during the remaining life of the structure such that subsequent repair work is expected. The condition of the structure will be reset to the initial condition after major repair work.

(4) Rehabilitation repair (Rep, 4): it restores the durability, structural strength, and function or appearance of the structure and improves the durability performance of the repaired structure or its components. Similar damage may occur later with lower probability. The condition of the structure will be reset to the initial condition after the work.

(5) Replacement (Rep, 5): it refers to the replacement of existing members, which may improve local capacity, durability of the structure, and so forth. The condition of the structure will be reset to the initial condition after the work.

11. Example of LCM Strategy of Steel Girders in Bridges

A simply supported composite steel girder bridge with rolled-beam stringers is adopted as an example to illustrate the proposed LCM strategy. The bridge has a simple span of 28.0 m with two-lane traffic in the same direction. 191 mm thick concrete deck slab with wearing surface at weight 122 kg/m^2 is supported by five rolled steel girders at size W36 × 245. Each girder has a welded cover-plate with size 386 × 10 mm thick under bottom flange. 865 mm high of concrete parapets are installed at both sides. The distance from each end of the cover plate to the adjacent bearing is 2.0 m. The cross section of the bridge is shown in Figure 4.

11.1. Loading Analysis

11.1.1. Dead and Superimposed Dead. It is assumed the dead and superimposed dead loads are normally distributed with coefficient of variation (C.O.V.) 0.1 [9, 10, 21]. The induced mean moments at the mid span and the end of cover plate and the induced mean shears at end supports are calculated and summarized in Table 3.

From Table 3, the maximum bending moment at mid span is 1,798 kNm, the bending moment at the ends of cover plates is 476.89 kNm, and the maximum shear load at supports is 257 kN for Girder Nos. 1 and 5. Whereas, for Girder Nos. 2, 3, and 4, the maximum bending at mid span is 1,400 kNm, the bending moment at ends of cover plates is 372 kNm, and the maximum shear load at supports is 200 kN.

11.1.2. Live and Impact. The live load on the bridge is assumed due to traffic only. Girders moments and shears summarized in Table 4 are calculated based on the survey results of Nowak [21] with literature references in which the average daily truck traffic (ADTT) is 5,000 with 66% of the cases that a truck was in the left lane, 33% of the cases that a truck was in the right lane, and 1% of the cases that trucks

Table 4: Moments and shears due to live load (M_{LL} and V_{LL}).

Girder no.	Position	Left lane			Right lane			Both lanes		
		Mean M_{LL} (kNm)	Mean V_{LL} (kN)	C.O.V.[1]	Mean M_{LL} (kNm)	Mean V_{LL} (kN)	C.O.V.[1]	Mean M_{LL} (kNm)	Mean V_{LL} (kN)	C.O.V.[1]
1	Mid span	2,333.46	—		567.56	—		2,720.61	—	
1	End of cover plate	815.24	—	0.24	198.29	—	0.33	950.50	—	0.23
1	Support	—	333.35		—	81.08		—	388.66	
2	Mid span	2,390.08	—		998.17	—		3,247.35	—	
2	End of cover plate	835.02	—	0.18	348.73	—	0.23	1,134.52	—	0.18
2	Support	—	341.44		—	142.6		—	463.91	
3	Mid span	1,786.97	—		1,735.61	—		3,356.65	—	
3	End of cover plate	624.31	—	0.19	606.37	—	0.19	1,172.71	—	0.19
3	Support	—	255.28		—	247.94		—	479.52	
4	Mid span	1,107.47	—		2,413.79	—		3,356.65	—	
4	End of cover plate	386.92	—	0.23	843.30	—	0.18	1,172.71	—	0.19
4	Support	—	158.21		—	344.84		—	479.52	
5	Mid span	654.47	—		2,440.12	—		2,920.77	—	
5	End of cover plate	228.65	—	0.27	852.50	—	0.22	1,020.43	—	0.21
5	Support	—	93.50		—	348.59		—	417.25	

[1]C.O.V.: coefficient of variation.

Table 5: Statistical parameters of random variables for the illustrative example.

Parameters	Materials	Variables	Mean (μ)	C.O.V.[1] (σ/μ)	Distribution	Sources
Corrosion deterioration rate	Carbon steel	A	70.6 μm	0.66	LN[2]	[5, 6]
		B	0.789	0.49		
		ρ_{AB}	−0.31	—	—	
	Weathering steel	A	40.2 μm	0.22	LN	
		B	0.557	0.10		
		ρ_{AB}	−0.45	—	—	
Corrosion Initiation	—	T_{CI}	15 yr	0.30	LN	[7]
Repaint duration	—	T_{REP}	20 yr	0.25		
Initial crack dimension	Structural steel	a_0	0.762 mm	0.5	LN	[14]
Crack growth constant		C	1.26×10^{-13}	0.63		
Crack growth exponent		m	3	—	Constant	[15]
Critical crack dimension		a_c	0.0254 m			
Fracture toughness		K_{IC}	43.97 MPa m$^{0.5}$	0.19	TN[3]	[10]
Compressive strength	Concrete	f_c	21 MPa[4]	0.19	LN	[9]
Yield stress	Steel	F_y	248 MPa	0.10		
Modulus of elasticity	Concrete	E_c	30,000 MPa	0.20	LN	[16]
	Steel	E_s	210,000 MPa	0.06		
Deck slab thickness	Concrete	t_c	191 mm	0.20	N[4]	[14]

[1]C.O.V.: coefficient of variation.
[2]LN: lognormal distribution.
[3]TN: truncated normal distribution.
[4]N: normal distribution.

Table 6: Deflection of steel girder bridge with cover plate.

Years	Nominal corrosion penetration (mm)			Nominal maximum deflection (mm)		
	Carbon steel without coat	Carbon steel with coat	Weathering Steel	Carbon steel without coat	Carbon steel with coat	Weathering steel
0	0.000	0.000	0.000	29.332	29.332	29.332
10	0.434	0.000	0.145	29.554	29.332	29.405
20	0.750	0.251	0.213	29.719	29.460	29.440
30	1.033	0.251	0.267	29.870	29.460	29.468
40	1.297	0.503	0.314	30.012	29.589	29.492
50	1.546	0.503	0.355	30.149	29.589	29.513
60	1.786	0.754	0.393	30.282	29.721	29.533
70	2.016	0.754	0.428	30.413	29.721	29.551
75	2.129	0.754	0.462	30.478	29.721	29.560

were in both lanes [9, 10, 14]. It is also assumed that the live load from vehicles is kept unchanged throughout the bridge's service life. To account for the dynamic effects of a vehicle riding over the bridge, an impact factor is adopted as a multiplier. The impact fraction of live load according to ASSHTO Specification (2004) [22] is 0.231. The moment and the shear due to impact will then be added on the live load schedule.

11.2. Service Life Prediction

11.2.1. Serviceability Limit: Deflection. Corrosion reduces the effective cross-section and occurs mainly at the web and the top of bottom flange of steel girders. Reduction of section area leads to weaken the flexural strength and the shear strength and increases the deflection of bridge deck under service loadings. According to the ASSHTO specification, the deflection limit due to live load plus impact is span/800 = 35 mm for typical highway bridges. In order to investigate the long-term effectiveness of different preventive measures of steel girders, three scenarios include carbon steel without protective paint, carbon steel with protective paint, and weathering steel are compared. The steel bridge is modeled against corrosion in marine environment based on the statistical parameters listed in Table 5. The nominal maximum deflections under three different scenarios are calculated according to (1) to (3) and summarized in Table 6.

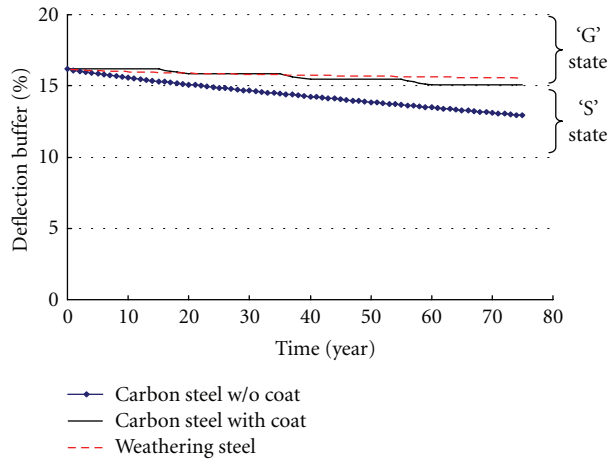

Figure 5: Deflection Buffer Over Life Span.

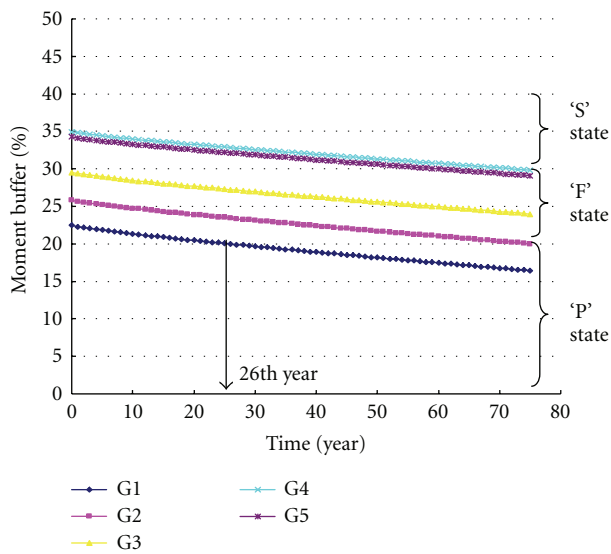

Figure 6: Moment buffer over life span of carbon steel girders.

The deflection buffer against time over the life span is plotted in Figure 5. Results show that the deflection for girders without any protective systems is the highest one while the deflection for girders made of weathering steel is the lowest. According to the proposed management strategy, throughout the 75 years service life, none of steel girder types falls below the fair condition state (i.e., "*F*"—State) in the Deflection Condition State Set (i.e., (13)). In other words, no R&R works for deflection is expected throughout the life span.

11.2.2. Ultimate Limits—Moment and Shear. It is assumed that R&R actions only be taken places when condition reaches "*P*" state range in accordance with (14) and (15). Simulated time to reach "*P*" state in terms of moment and shear buffer limits at 80 percentile value are plotted in Figures 6 and 7 respectively. Concerning the moment condition, result shows that Girder No. 1 will reach the moment buffer limit in the 26th year and the other girders will remain "*F*" state and "*S*" state throughout the life span. As to the shear condition, results demonstrate all girders will remain "*G*" state that means shear stress causes insignificant impact to girders.

11.2.3. Fatigue Damage Limit. The maximum live load model developed above is insufficient to determine the effective range of the stress intensity factors. For fatigue analysis, the loading effects are modeled by the fatigue truck provided in the American Association of State Highway and Transportation Officials (AASHTO) specification with gross weight 240 kN. Each truck passage is assumed to cause one stress cycle only. Under repeated tensile stresses, fatigue cracks may form at the end weld of the cover plate and penetrate into the bottom flange. Under given bending moment in a girder, the critical fatigue stress is located in the bottom flange at the end of cover plate such that it should

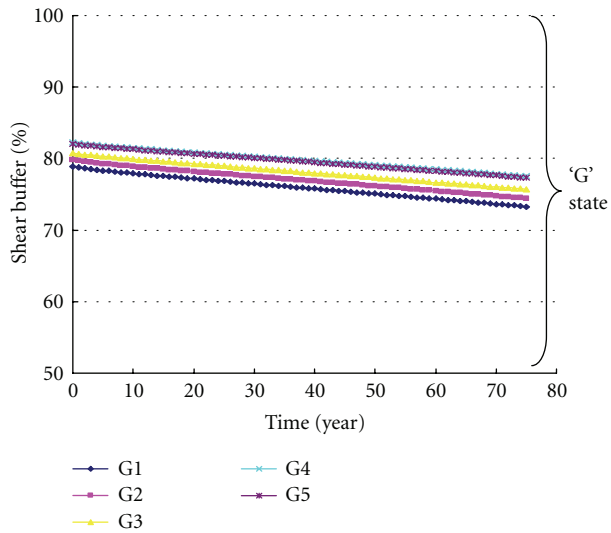

Figure 7: Shear buffer over life span of carbon steel girders.

Table 7: Time (year) to "*P*" state of fatigue buffers.

Girder no.	Time
G1	48
G2	43
G3	47
G4	50
G5	62

be calculated based on the cross section of the girder without cover plate. The service life of each girder is simulated based on the statistical parameters in Table 5 and the simulated time for each girder to reach its "*P*" state is summarized in Table 7.

Results show that girder no. 2 is the most sensitive girder to the fatigue damage, and it will reach the fatigue limit in the 43rd year.

12. Discussion

There are two important messages incurred from the example: (1) fatigue damages may not always dominate. Conventional LCM is in accordance with fixed predefined limit states either the serviceability limit or the ultimate limits or the fatigue limit. However, it may not be always valid that one limit dominates over one another or vice versa under all situations. Results demonstrate that the earliest simulated replacement or rehabilitation time for girders under bending stress is in the 26th year while the earliest simulated repair time for girders under fatigue damages is in the 43th year and (2) service lives of each girders are various. Conventional management approach treats all girders on equal ground. However, some girders may deteriorate faster than expected and some may not. In the case of fatigue damages, girder no. 5 has the longest life while girder no. 2 has the shortest one. Concerning the bending failure, the most critical one is girder no.1 while the least are girder nos. 4 and 5.

Service life prediction models integrated into the management strategy could provide a better picture for the stakeholders to identify the sensitive steel girder under corrosion deterioration or fatigue damages. Other factors affecting the condition of steel girders are neglected in the paper. However, steel components damages are definitely not conclusive to these two types.

Moreover, proposed acceptable limits can be further tied into reliability index if sufficient statistical information of the parameters in deterioration models is available. The rehabilitation time can be determined by the predefined reliability index value. Even though the reliability assessment has not been incorporated in this paper, the proposed LCM concept is not affected.

Furthermore, owing to the lack of sufficient cost data, the paper only addresses the LCM strategy and the methodology of service life prediction models being integrated into the LCC model. If sufficient financial data is available, it could provide further decision making information for selecting the most appropriate management strategy by LCC comparison.

13. Further Research Approach

Further research approach on the proposed flow for LCM strategy on steel girder in bridges as shown in Figure 8 can be conducted to test and verify its practicability. The proposed verification approach is categorized into 7 stages from Design Stage to Analysis Stage. However, some key important data should be obtained in the investigation stage prior to the LCC analysis process.

(1) Cost Data. Collection of possible future cost data is necessary to provide further decision making information for selecting the most appropriate management strategy.

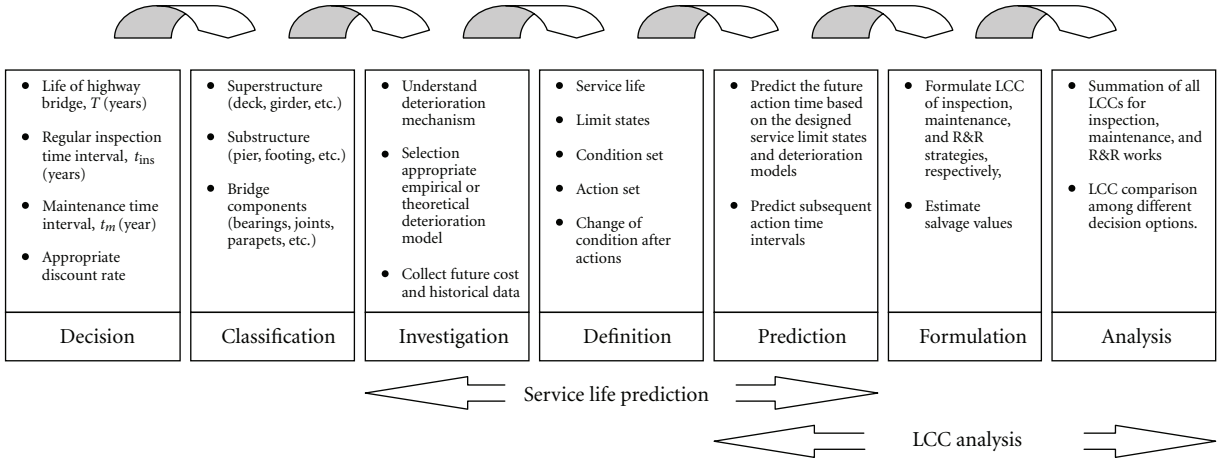

Figure 8: Suggested LCM flows for inspection, maintenance, and replacement and rehabilitation Works.

The cost of different types of future work should comprise inspection, operation, maintenance, management, repair, rehabilitation, replacement, demolition and failure.

(2) Historical Data. Collection of historical repair or rehabilitation data can be used to estimate the probability of any occurrence in case the future action time could not be predicted by appropriate deterioration models.

(3) Deterioration Mechanism. Steel component damage is definitely not exclusive to corrosion deterioration and fatigue damage. In the study, other factors affecting the condition of the steel girders are neglected. In fact, the bearing capacity of the steel girders and deterioration of other bridge components such as bridge bearings, expansion joints, and profile barriers could be incorporated into the LCM strategy. The proposed LCM strategy can be further enlarged.

14. Conclusion

Technical review on structural steel corrosion and fatigue models is addressed in this paper. However, in order to properly develop a LCM strategy for steel bridges in particular regions, it is necessary to consider some adjustments of parameters on the service life prediction models to suit the regional characteristics and needs. Further investigations on different empirical models under Hong Kong environments will be undertaken to identify the most appropriate empirical models for service life prediction purpose.

This paper proposes an integrated LCM framework assisting stakeholders to appropriately and reasonably prioritize their future maintenance-related works on steel girders in their bridge stocks such that stakeholders can better allocate the limited resources. In the framework, corrosion deterioration and fatigue damage prediction models are mapped with girders' performance conditions on deflection, ultimate moment and shear capacities, and fatigue strength limit simultaneously. The illustrative example on the proposed strategy on steel girders is also provided to demonstrate its applicability.

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