Strength Degradation Analysis of an Aging RC Girder Bridge Due to Existing Cracks

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ABSTRACT. Aging reinforced concrete (RC) bridges require regular evaluation of structural integrity taking into account the actual deterioration of materials. This study selected a multi-span RC girder bridge with a service life of almost 80 years and used FE analysis to examine its severely cracked central girder, focusing on the effects of existing cracks on the structural behavior and load-carrying capacity. The study shows that depending on the type of cracking, the existing cracks may have serious effects on the structural integrity of the bridge. For a simply- supported girder, while the effects of central cracks can be ignored in principle, shear cracks may greatly reduce the load-carrying capacity of the bridge and may even cause brittle failure of the structure. Strength evaluations are carried out using the load and resistance factor design (LRFD) method of AASHTO and related issues are discussed.

INTRODUCTION

Aging reinforced concrete (RC) bridges categorized as requiring extensive maintenance and repair are increasing in number as the design service life of these old structures is either approaching or has already passed. Many of those that were built during the first half of the last century, have sustained severe concrete cracking and rebar corrosion and have undergone major repairs and renovation [1, 2]. In diagnosing potential structural problems for these aging bridges, crack analysis is still subjected to the old prejudice of possessing too many uncertainties, and therefore, has not been fully utilized for analyzing bridge safety. Realistic structural problems that involve concrete cracking are often simplified under assumed failure modes with simple mechanics models that ignore tensile stress in concrete, thus nullifying the necessity for crack analysis. However, these approaches may not always be valid, as in the case of complex failure modes.

The past decade has witnessed remarkable progress in computational fracture mechanics of concrete [3]. In this study, a recently- inspected multi-span RC girder bridge with a service life of nearly 80 years was selected for crack analysis of its severely- cracked central span, focusing on the various structural effects of existing cracks on the load-carrying capacity of the girder. To provide a uniform basis for comparison, the strength of the bridge is calculated using the load rating factor method of AASHTO (American Association of State Highway and Transportation Officials) [4]. Based on field surveys of the bridge, the existing cracks are grouped into two categories: central cracks and shear cracks. As the field measurements did not yield much quantitative information on the sizes of these cracks, numerical studies are carried out to investigate how the depth and location of these cracks can affect the strength of the girder. Hopefully, this will yield useful information on the various structural effects caused by concrete cracking and how these effects should be assessed in the maintenance works for aging RC bridges.

OUTLINE OF THE BRIDGE

Figure 1 shows a partial profile of the seven-span RC girder bridge with a service life approaching 80 years. Visual structural inspections as part of the maintenance work for the bridge have been conducted at regular intervals of several years using non-destructive techniques such as radar and electronic measuring devices. Based on the past maintenance records, the bridge had sustained a wide range of material and structural deterioration with concrete cracking, cover spalling and rebar corrosion being reported before major renovation work was carried out 15 years ago. Since then, the aging bridge has been reinforced with steel I-beams installed in its suspension spans, and thick steel plates fixed to the bottoms of all its RC girders.

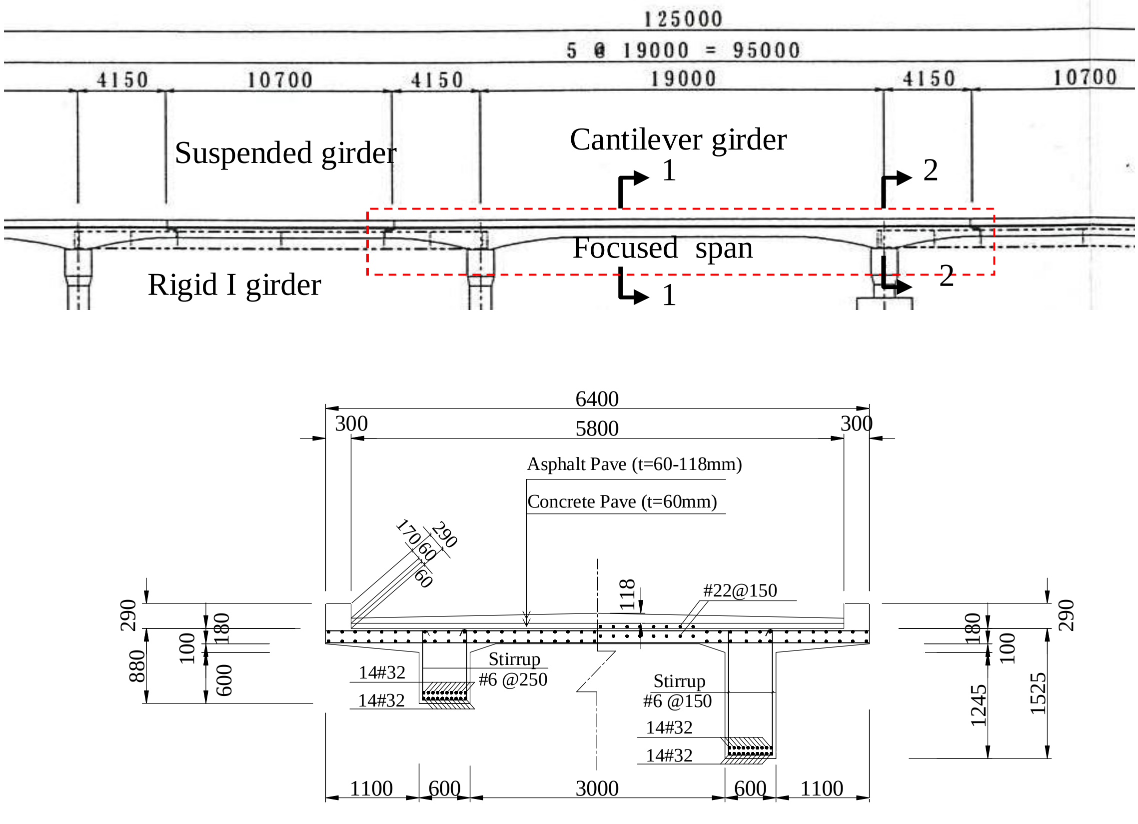


Fig. 1 Bridge profile and cross sections

A new structural safety study using recent field inspection data has cleared the bridge for various strength requirements by the Specifications for Highway Bridges of JRA (Japan Road Association) and JSCE (Japan Society of Civil Engineers) [5, 6], which demands the reduction of rebar areas according to the severity of concrete cracking. Even though the present codes do not require direct inclusion of concrete cracking in strength evaluations of RC bridges, there are legitimate concerns regarding the possible adverse structural effects of existing cracks, especially when shear strength is in question. Motivated by these concerns, the central cantilever girder of the bridge with an extensive cracking record is studied by FE analysis, focusing on the structural effects of existing cracks on the load-carrying capacity of the beam. To reflect the general structural characteristics of RC girder bridges, only the original structural components of the central span are modeled, excluding the steel plate attached to the bottom of the girder at the time of renovation.

FE MODELING

Concrete and Rebar Modeling

Due to the symmetrical conditions of the bridge cross section as shown in Fig. 1, only one girder and half of the upper deck are modeled. Figure 2 shows a two-dimensional FE model of the central span, and numerical analyses are carried out using the DIANA commercial FE software package [7]. In this simplified modeling approach, the plane-stress condition is assumed for the girder and upper deck separately, each with its own width. The material behavior of concrete is modeled using the total strain crack model in which a parabolic curve for compressive crushing and a tension softening curve for tensile cracking are employed, as presented in Fig. 3. The embedded-bar element and the grid-reinforcement element in DIANA are used to model the longitudinal steel bars and stirrup bars, respectively, and the rebar material properties are assumed to be elastic and perfectly-plastic. The assumption of a rigid bond between the reinforcement and the concrete is applied.

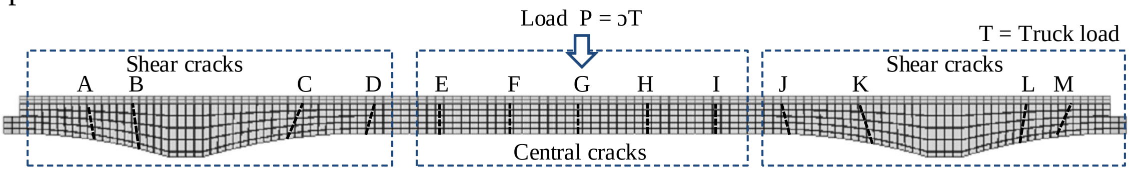


Fig.2 FE model

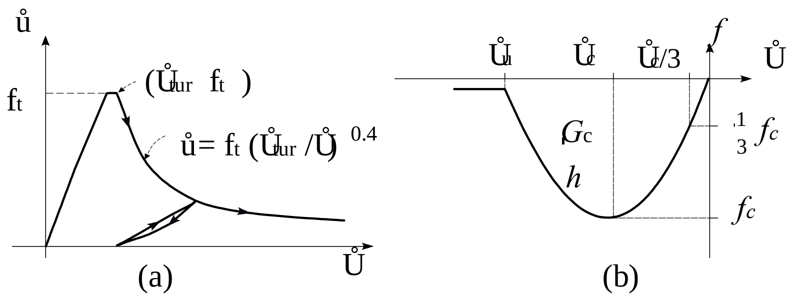


Fig. 3 Stress-strain curves of reinforced concrete [(a) tensile stress-strain curve and (b) compression stress-strain curve]

Modeling of Existing Cracks

Based on the drawings produced from field inspections, existing cracks are roughly divided into two groups depending on their location in the girder, i.e., central cracks in the middle region of the span and shear cracks close to the piers, as shown in Fig. 2. As field surveys on cracks usually do not reveal the scale of inner cracking, the following numerical studies assume various values for the sizes of existing cracks in each group, starting from the minimum size of one mesh to the maximum size of the girder height. An existing crack is modeled discretely using structural interface elements, which allow an initial crack to open up when the normal traction on the surface of the interface element becomes tensile.

Boundary and Load Conditions

Field surveys did not fully clarify the support conditions at the two piers, and there seemed to be no mechanical bearings installed between the girder and the pier. Therefore, both pin-roller and pin-pin support conditions, are assumed for numerical studies. With the pin-pin supports (simplified as HFF), all vertical and horizontal movements of the girder are fixed at the two piers. With the pin-roller supports (HFM), however, horizontal movement is allowed at one of the piers. As for the load conditions, besides the dead loads, the live loads are represented by the simple truck load specified in the JRA design standard.

ANALYTICAL METHOD FOR LOAD CAPACITY EVALUATION

The AASHTO Manual for Bridge Evaluation [8] presents an analytical method for evaluating the load capacity of in-service bridges, based on the load and resistance factor design (LRFD) method. The general load rating equation is expressed as

$$RF = \frac{C - (\gamma_{DC})DC - (\gamma_{DW})DW \pm (\gamma_P)P}{(\gamma_{LL})(LL + IM)} \tag{1}$$

where RF = rating factor; C = ū $_{C}$ū $_{S}$R$_{n}$; R$_{n}$ = nominal member capacity; ū $_{C}$ = condition factor; ū $_{S}$ = system factor; DC = dead load effect due to structural components and attachments; DW = dead load effect due to wearing of surface and utilities;ɔ$_{DC}$ = dead load factor; P = permanent load effect other than dead loads; LL = nominal live load effect caused either by truck or lane loading; IM = dynamic load allowance; ɔ$_{DW}$ = dead load factor; ɔ$_{P}$ = permanent load factor; and ɔ$_{L}$ = live load factor. In Japan, according to the JRA design standard, bridge evaluation is performed using two methods: for performance check, the allowable stress design (ASD) method is used, and for strength evaluation, the limit state design (LSD) method is employed. In the LSD, the ultimate collapse loads are given by three load combinations: (1) 1.3 × dead load + 2.5 × live load; (2) 1.0 × dead load + 2.5 × live load; and (3) 1.7 × dead load + 1.7 × live load. By unifying the expressions for the dead load effects of DC and DW in Eq. (1) and omitting the permanent load effect P, Eq. (1) can be rewritten as:

$$(\gamma_{LL})\text{RF} \quad = \frac{C-(\gamma_{DL})DL}{(LL+IM)} \qquad (2)$$

where (ɔ$_{LL}$)RF = nominal rating factor (live load capacity); DL = total dead loads; and ɔ$_{DL}$ = dead load factor (= 1.0, 1.3, 1.7 in JRA; = 1.3 in AASHTO).

RESULTS OF NUMERICAL STUDY AND DISCUSSION

Numerical analyses are carried out on the target girder and numerical results are presented in terms of the structural failure behavior and load-carrying capacity. Herein, the load-carrying capacity is represented by the ratio of the maximum live load to the design live load, which just corresponds to the nominal load rating factor defined by Eq. (2). Material properties employed in the numerical analyses are summarized in Table 1.

Table 1 Material properties

Concrete	Compressive strength	f_{ck}	22.2	N/mm^2	Surveyed
	Tensile strength	f_{ct}	1.8	N/mm^2	JCI
	Young's modulus	E_c	24100.0	N/mm^2	
	Fractural energy	G_f	82.2	N/mm	JCI
Reinforcement (SR235)	Yield strength	f_y	235.0	N/mm^2	
	Young's modulus	E_s	210000.0	N/mm^2	

Structural Failure Behavior

Figures 4(a) and 4(b) illustrate the load-displacement relationship in a simply- supported central girder (HFM) under three-point bending with the presence of only central cracks and only shear cracks, respectively. The corresponding results at structural failure on crack

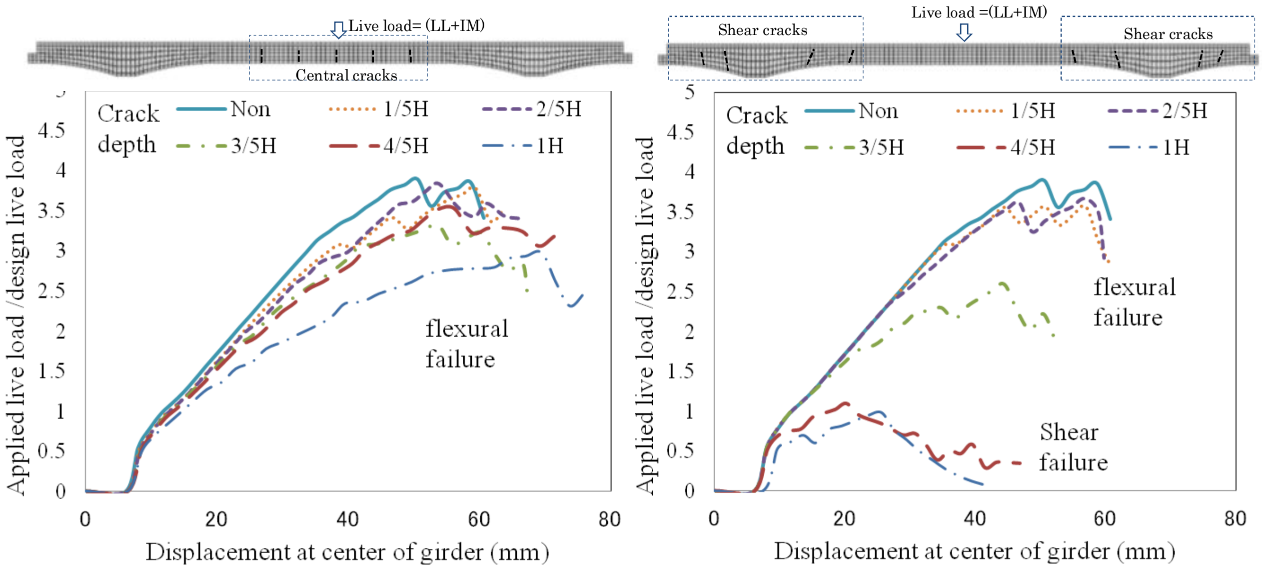


Fig.4 Load-displacement relations (ɔ$_D$ = 1.0) [(a) the case of flexural crack propagation and (b) the case of shear crack propagation]

distribution and stress contours of the rebar and stirrup with the size of the initial cracks being set at 4/5H (H = height of girder) are presented in Figs. 5 and 6. Note that in these analyses the dead load factor ↄ$_{DL}$ is assumed to be 1.0.

As shown in Fig. 4(a), with the presence of the central cracks there is no significant change in the general structural behavior of the girder, as the size of these initial cracks varies from small to large and the failure modes are typically flexural failures. On the other hand, if the existing cracks in the girder are of the shear type, then depending on the scale of these cracks, the failure mode of the girder can change considerably. As is clearly shown in Fig. 4(b), when these shear cracks are small, the beam typically fails in a flexural failure mode, but as the cracks become larger, the beam then fails in a significantly different shear mode that is accompanied by very limited deformation, implying brittle failure.

Other interesting features and differences of these two failure modes can be found in one of the cases studied (assuming the size of relevant cracks equal to 4/5 the height of the girder) with the corresponding numerical results on crack distribution and stress contours of the rebar and stirrup being shown in Figs. 5 and 6, respectively. As these results are easy to comprehend, further discussions are omitted here.

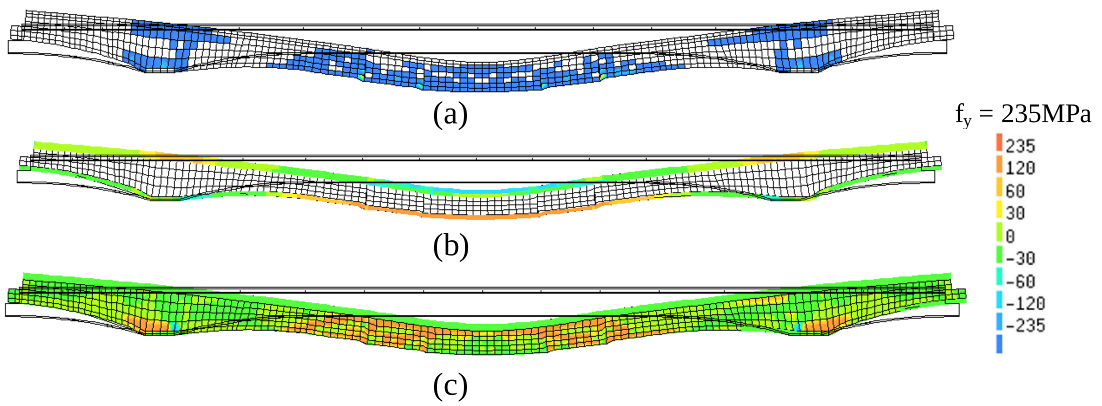


Fig. 5 Features of flexural failure (size of initial central cracks = 4/5 H) [(a) crack distribution contour, (b) stress contour of steel bar, and (c) stress contour of stirrup]

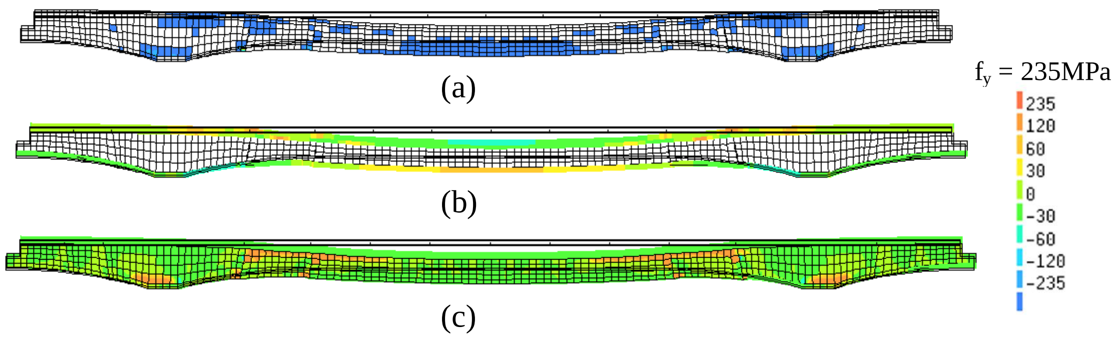
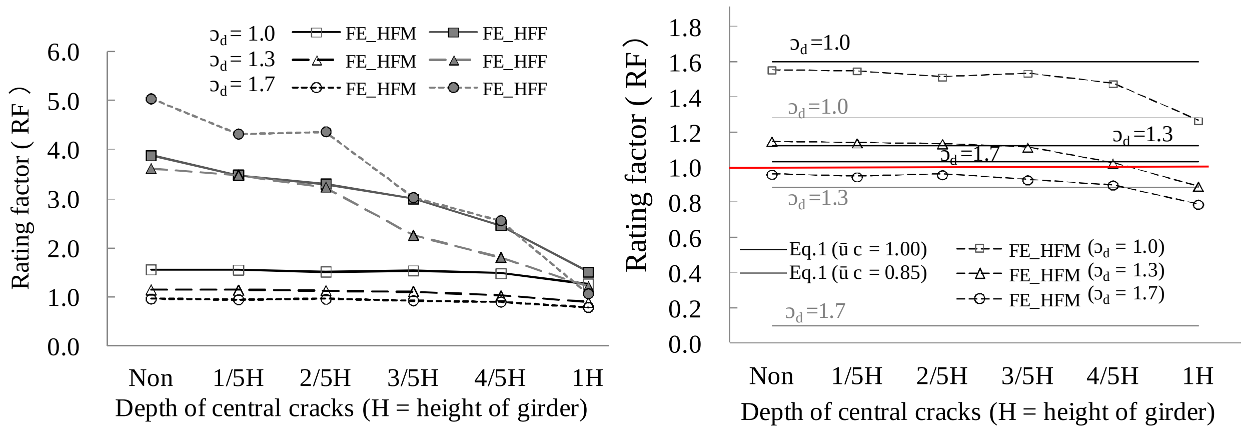


Fig. 6 Features of shear failure (size of initial central cracks = 4/5 H [(a) crack distribution contour, (b) stress contour of steel bar, and (c) stress contour of stirrup]

Load-Carrying Capacity

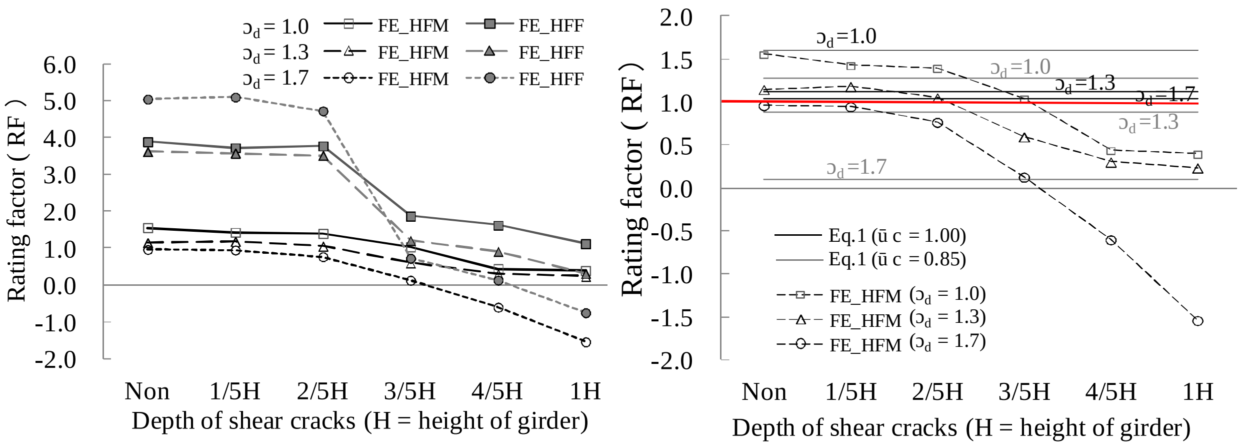
Rating factors representing the load-carrying capacity of the girder are computed using the nominal rating factors obtained numerically as the ratio of the maximum live load to the design live load, assuming various sizes for the existing cracks. Figures 7 and 8 summarize the relationship between the rating factor and the depth of existing cracks. Analytical results obtained by the LRFD method of Eq. (2) are also plotted in these figures. As shown in the figures, the effects of the existing cracks on the load-carrying capacity depend not only on the type of cracks, but also on the support conditions.

Under the simple support condition (HFM), the strength degradation of the girder due to the existence of central cracks is generally quite limited, as clearly shown in Fig. 7. While the degradation effects on the girder strength can be completely ignored before the central



(a) (b)

Fig. 7 Relationship of rating factor and size of central cracks [(a) FE analysis results and (b) comparison between FE analysis (HFM) and LRFD results]



(a) (b)

Fig. 8 Relationship of rating factor and size of shear cracks [(a) FE analysis results and (b) comparison between FE analysis and LRFD results] cracks reach half the girder height, the maximum degradation rate is still less than 20 % as these cracks nearly penetrate the girder. On the other hand, the reduction effects of shear cracks on the strength of the girder can be serious. As illustrated in Fig. 8, as the shear cracks reach a size that is 2/5 of the girder height, the reduction rate can be as great as 20 %, beyond which structural failure can occur as the required load rating factor of Eq. (2) (where the minimum value of ū$_C$ = 0.85 is adopted) is no longer satisfied. As studies on the effects of boundary conditions and on how crack analysis can help determine the condition factor ū$_C$ in Eqs. (1) and (2) are still preliminary, further discussions on these topics are omitted.

CONCLUSIONS

The following conclusions are drawn from the study:
(1) An analytical approach for assessing the structural integrity of aging RC bridges using the load rating factor of AASHTO has been proposed. Based on numerical studies, it is considered that the method may provide an effective way of assessing an aging RC bridge based on the crack conditions in concrete and the corrosion of steel reinforcement, as well as other structural features including the support conditions.
(2) The presence of central cracks in the RC girder studied has no significant effect on its general structural behavior, but the existence of shear cracks in the girder can have a serious effect. When these shear cracks are small, the RC girder typically fails in a flexural failure mode, but as the cracks become larger the beam then fails in a shear mode with very limited deformation, implying brittle failure.
(3) Under the simple support condition, the degradation of girder strength caused by the existence of central cracks is rather small and can be ignored in most cases. On the other hand, shear cracks can greatly reduce the strength of the girder. Even small shear cracks can significantly reduce the strength of the girder, while large shear cracks may trigger brittle structural failure, seriously endangering the structural safety of the girder.

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