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RESEARCH

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Seismic Performance Assessment of Deteriorated Two-Span Reinforced Concrete Bridges

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Abstract

This paper presents a nonlinear analysis procedure for the seismic performance assessment of deteriorated reinforced concrete bridges using a modified damage index. A finite-element analysis program, RCAHEST (Reinforced Concrete Analysis in Higher Evaluation System Technology), is used to analyze deteriorated two-span simply supported reinforced concrete bridges. The new nonlinear material models for deteriorated reinforced concrete behaviors were proposed, considering corrosion effects as shown in a reduction in reinforcement section and bond strength. A modified damage index aims to quantify the seismic performance level in deteriorated reinforced concrete bridges. Several parameters of two-span simply supported deteriorated reinforced concrete bridge have been studied to determine the seismic performance levels. The newly developed analytical method for assessing the seismic performance of deteriorated reinforced concrete bridges is verified by comparison with the experimental and analytical parameter results.

Keywords: deteriorated, reinforced concrete, bridge, seismic performance, modified damage index

1 Introduction

Many existing reinforced concrete structures deteriorate owing to little attention to durability issues and considerable resources are expended to rehabilitate and repair deteriorating concrete bridge structures.

The lifetime seismic performance assessment of reinforced concrete bridges in aggressive environment should account for both the diffusion process of aggressive agents, such as chlorides, and the mechanical damage induced by diffusion (Fernandez & Berrocal, 2019; Ramseyer & Kang, 2012; Song et al., 2019; Tapan, 2007; Xu, Cai, et al., 2021; Xu, Feng, et al., 2020, 2021; Xu, Wu, et al., 2020; Yang & DeWolf, 2002).

It is generally recognized that cracks provide easy access to ingress of chlorides in concrete and hence, the initiation of corrosion of steel in cracked concrete occurs at early stage. Corroded reinforcement in deteriorated structures that are subjected to seismic loads can decrease their robustness and ductility significantly, because the ultimate strain and elongation of the reinforcing steels are reduced. The corrosion process causes not only a reduction in the steel mass, but also a loss of ductility of the material that can lead to brittle failures of concrete members (Al-Harthy et al., 2011; Cairns et al., 2008; Du et al., 2005; Hanjari et al., 2011; Lignola et al., 2010; Morga & Marano, 2015; Shaikh, 2018).

The purpose of this study is to provide knowledge for analytical seismic performance evaluation of deteriorated reinforced concrete bridges using a modified damage index. Experimental evaluation of seismic performance of these reinforced concrete bridges is time consuming and costly (Kim, 2019).

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In this study writer has developed a new analytical seismic performance assessment method for deteriorated reinforced concrete bridges. A developed program RCAHEST (Reinforced Concrete Analysis in Higher Evaluation System Technology) was used (Kim, 2012, 2019; Kim et al., 2003, 2005, 2007).

This study also presents a new improved seismic performance assessment method that has several advantages. The author proposes new deteriorated material models to predict the seismic behaviors of deteriorated reinforced concrete bridges. The modified damage index was verified from the parametric study of deteriorated two-span simply supported reinforced concrete bridge using nonlinear finite-element analysis.

To assess the ability of the RCAHEST program to predict the seismic performance of deteriorated reinforced concrete bridges, analytical results were compared with the experimental and analytical parameter results.

2 Reinforced Concrete Analysis in Higher Evaluation System Technology (RCAHEST)

For an accurate evaluation of the inelastic behavior of deteriorated reinforced concrete bridges, constitutive modeling and three-dimensional finite-element analysis are required. However, difficulties in developing a reliable three-dimensional constitutive model and the extensive number of calculations required pose several problems in the actual problem application (Kim, 2012, 2019). Therefore, a two-dimensional material model of deteriorated reinforced concrete bridges is used in this study. The model was analyzed using general-purpose finite-element software, RCAHEST (Kim, 2012, 2019; Kim et al., 2003, 2005, 2007). RCAHEST is a finite-element analysis program used for analyzing reinforced and prestressed concrete structures. The structural element library RCAHEST is built around the finite-element analysis program shell named FEAP (Taylor, 2000).

2.1 Overview

The models for material nonlinearity include tensile, compressive, and shear models for cracked concrete and a model of reinforcing steel, where the smeared crack approach is incorporated.

Concrete models may be divided into isotropic uncracked concrete models and cracked concrete models. For cracked concrete, the three models are for depicting concrete behavior in the direction perpendicular to the crack plane, in the direction of the crack plane and in the shear direction at the crack plane (see Fig. 1). The basic model adopted for crack representation is the nonorthogonal fixed crack approach of the smeared crack concept.

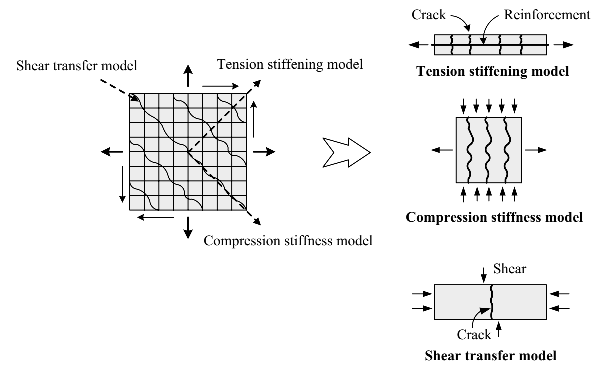

Fig. 1 Construction of cracked concrete model.

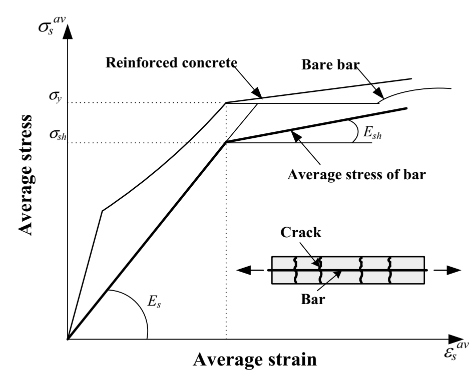

Fig. 2 Model for reinforcing bar in concrete.

The post-yield constitutive relationship of the reinforcement in concrete takes into account the bond characteristics, and is a bilinear model, as shown in Fig. 2 (Kim et al., 2003). The transverse reinforcing bars confine the core concrete, suppress the buckling of the longitudinal reinforcing bars and improve the ductility capacity of the unconfined concrete. In this study, writer basically adopted the model proposed by Mander et al. (1988).

Fatigue damage of reinforced concrete bridge columns under seismic load seems inevitable, and the fatigue damage may be characterized as low cycle fatigue of reinforcing bars and concrete strength deterioration (Kim et al., 2005).

A complete description of the nonlinear material model is provided by authors (Kim, 2012, 2019; Kim et al., 2003, 2005, 2007).

2.2 Deterioration Modeling in Reinforced Concrete

The developed degradation model takes into account uniform and localized corrosion and includes the reduction of cross-sectional area and bond strength of corroded bars.

A model proposed by Bhargava et al. (2007) to evaluate of corroded reinforcing bars and concrete deterioration was basically adopted in the finite-element model.

Bhargava et al. (2007) carried out experimental tests on corroded reinforced concrete specimens based on pullout tests and came about with the follow equations:

$$R = 1.0 \text{ for C} \leq 1.5\% \tag{1}$$

$$R = 1.192e^{-0.117C} \text{ for C} > 1.5\% \tag{2}$$

$$C = \frac{\Delta W}{W} \times 100 \tag{3}$$

where R is the ratio of bond strength of corroded reinforcing bar to bond strength of non-corroded reinforcing bar, C is the percentage of corrosion level, ΔW is the average mass loss of corroded reinforcing bars and W is the mass of non-corroded reinforcing bars.

The area A_{sc} of the corroded reinforcing bar can be represented as follows:

$$A_{sc} = A_s(1 - 0.01C) \tag{4}$$

where A_s is the area of the non-corroded reinforcing bar.

2.3 Seismic Performance Assessment Using a Modified Damage Index

An analytical evaluation method using a damage index was first proposed to assess damage states and seismic performance levels of solid reinforced concrete columns. Explicit descriptions of the different seismic performance levels are defined to employ specific engineering criteria (Kim et al., 2007).

In this study, a damage index was modified from the parametric study of deteriorated reinforced concrete bridge using nonlinear finite-element analysis. A parametric study was carried out to investigate the reduction of cross-sectional area and bond strength of corroded bars.

Seismic performance level	Service	Repair	Damage State	Damage Index
Fully operational	Fully service	Limited epoxy injection	Hairline cracks	≤ 0.1
Delayed operational	Limited service	Epoxy injection, Concrete patching	Open cracks, Concrete spalling	≤ 0.4
Stability	Not useable	Replacement of damaged section	Bar buckling/Fracture, Core crushing	≤ 0.75

Material	Type of failure	Failure criterion (ε_{cu} or ε_{tu})	Damage index ($DI_{compressive}$ or $DI_{tensile}$)
Concrete	Compressive and shear	$0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{sm}}{f'_{cc}}$	$1 - \Phi_c\left(\frac{2\varepsilon_{cu} - \varepsilon_{cs}}{2\varepsilon_{cu}}\right)^2$
Steel	Tensile	0.10	$1.20\left(\frac{\varepsilon_{ts}}{2\Phi_r\varepsilon_{tu}}\right)^{0.67}$

Fig. 3 Seismic performance assessment using a damage index (Kim, 2019).

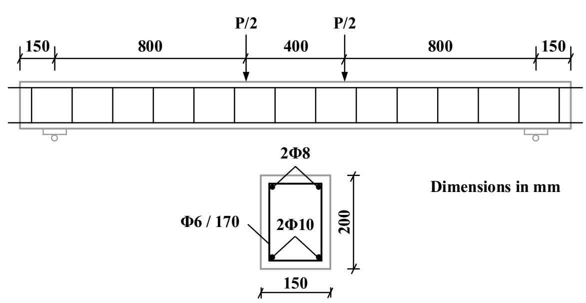

Fig. 4 Deteriorated reinforced concrete beams.

Fig. 3 provides such descriptions that might be associated with the three seismic performance levels of deteriorated reinforced concrete bridge. For the fully operational seismic performance level means almost undamaged and repair is not required. For the delayed operational seismic performance level means impairs its full use and might require repair. Finally, for the stability seismic performance level means severe damage requiring partial or complete

Table 1 Properties of deteriorated test specimens.

Specimen	Days	Corrosion penetration	
		Tensile bars	Compressive bars
111	–	–	–
114	117	0.45	0.52
115	101	0.36	0.26

f_{ck} = 50 MPa

$\Phi 6 f_{sy}$ = 626 MPa; $\Phi 8 f_{sy}$ = 615 MPa; $\Phi 10 f_{sy}$ = 575 MPa.

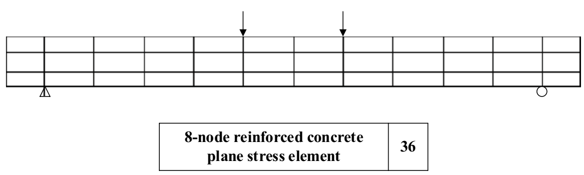

Fig. 5 Finite-element model for deteriorated reinforced concrete beams.

replacement. A complete description of the seismic performance assessment using a damage index is provided by Kim et al. (2007) and Kim (2019).

3 Verification of the Developed Deteriorated Material Model

The deteriorated reinforced concrete beams tested by Rodriguez et al. (1997) were used to validate the proposed deteriorated nonlinear material model.

The reinforced concrete beams were cast adding calcium chloride to the mixing water, subjected to an accelerated corrosion process with a current density of 100 mA/cm^2 and finally loaded up to failure. Fig. 4 shows the beam specimens details and reinforcement arrangements. The mechanical properties of the specimens are listed in Table 1.

Fig. 5 shows the finite-element discretization and the boundary conditions for deteriorated reinforced concrete beam specimens. Mesh size sensitivity analysis is also carried out. The cross-sectional area of the corroded reinforcing bars is computed by the proposed corrosion penetration model.

The mid-span load–deflection response for non-deteriorated beam specimen is shown in Fig. 6a. Fig. 6b, c also shows the experimental and analytical load–deflection relation of deteriorated beam exhibiting flexural failure with rupture of the tensile reinforcing bars.

In predicting the results of the deteriorated reinforced concrete beams, the mean ratios of experimental-to-analytical maximum strength were 1.03 at a CV of 2%. The good agreement between experimental and

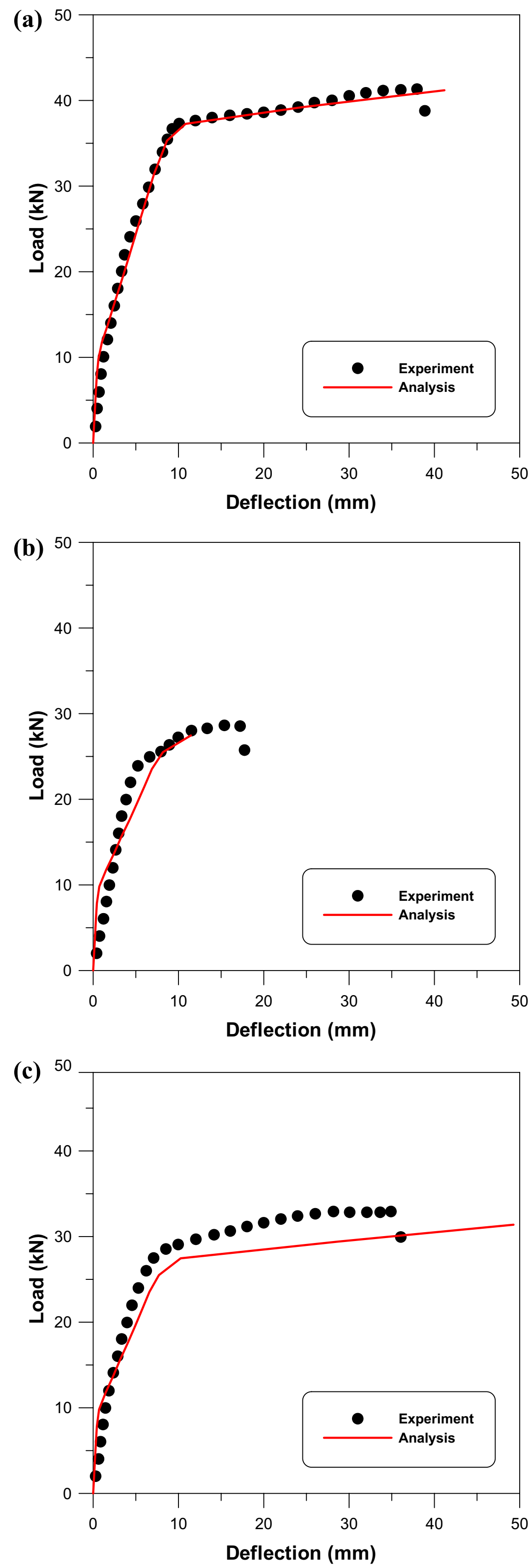

Fig. 6 Load–deflection curves: **a** Specimen 111, **b** Specimen 114 and **c** Specimen 115.

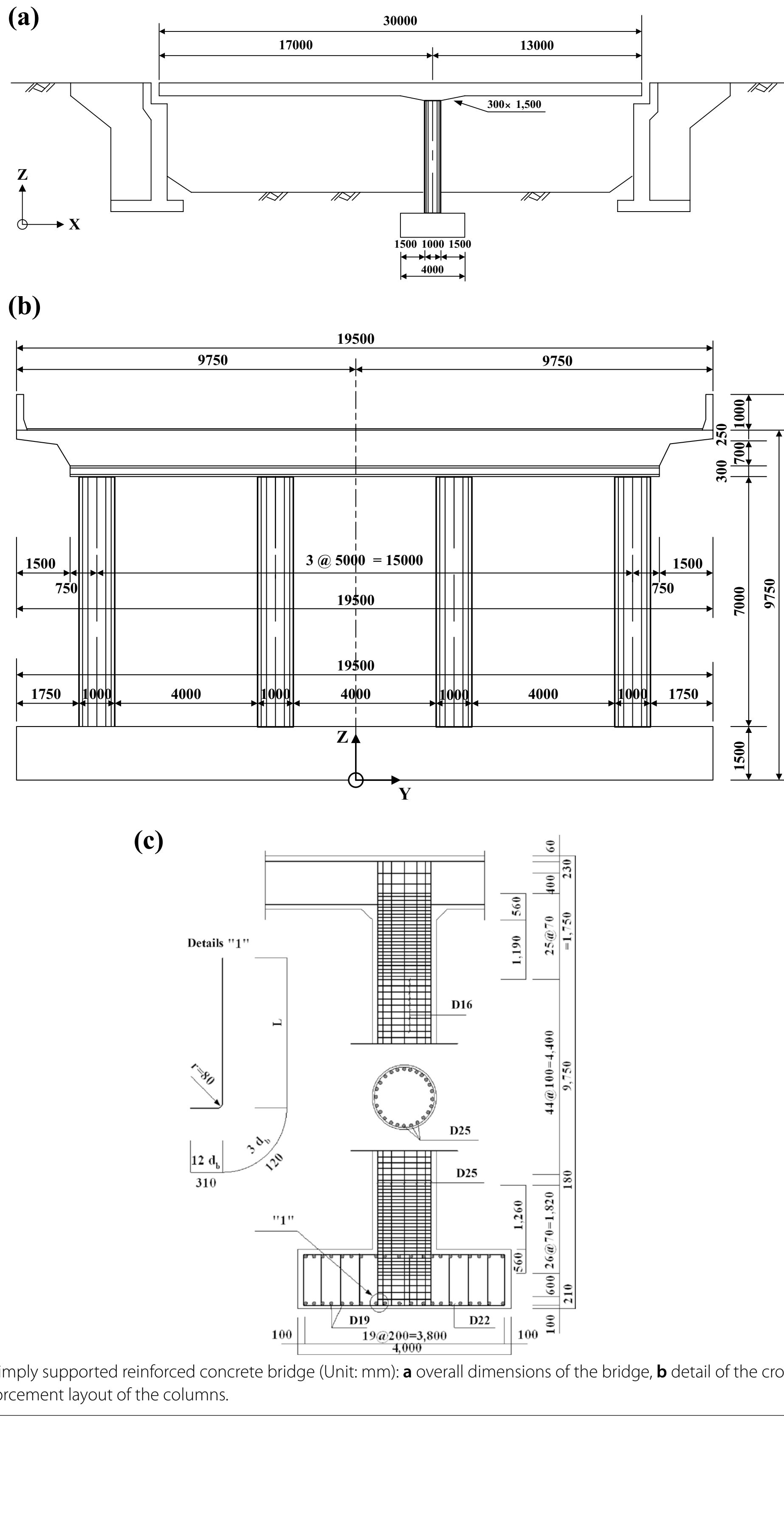

Fig. 7 Two-span simply supported reinforced concrete bridge (Unit: mm): **a** overall dimensions of the bridge, **b** detail of the cross-section and **c** details of the reinforcement layout of the columns.

analytical results demonstrates the accuracy of the proposed deterioration model. However, the analytical results with proposed model cannot capture the softening responses of the components due to the load control.

4 Application to a Two-Span Simply Supported Reinforced Concrete Bridge

In this section, a parametric study of the two-span bridges is conducted to provide a better understanding of the seismic performance of deteriorated reinforced concrete bridges.

4.1 Two-span Simply Supported Reinforced Concrete Bridge

An application example shown in Fig. 7 was designed to obtain seismic performance data of reinforced concrete bridges having details typical of those in use in regions of moderate seismicity (Korea Expressway Corporation, 2000).

The total length of the bridge slab is 30 m, with spans of 17 m and 13 m. The height of the bridge columns is 9.75 m. Fig. 7 shows the overall dimensions of the bridge: area of slab A_{dx}= 17.475 m^2; moment of inertia in the bridge axis direction I_z= 1.299 m^4; moment of inertia in the direction perpendicular to the bridge axis I_y= 496.340 m^4; torsional moment of inertia J= 4.973 m^4. The columns have circular cross section with diameter $\Phi = 1000$ mm and are reinforced with D25 longitudinal bars.

The constitutive laws are also defined by the following nominal values: concrete compressive strength f_{ck}= 27 MPa; steel yielding strength f_{sy}= 400 MPa. Seismic nonlinear analysis is carried out by considering a uniform gravity load of 491 kN/m, including self-weight and dead loads applied on the slab. The two-span simply supported reinforced concrete bridge was designed considering current recommendations and requirements for shear and confinement (AASHTO, 2012; CEN, 2004; MCT, 2015).

Non-linear time-history analyses are performed for a set of artificial earthquakes generated to comply with the elastic response spectrum given by MCT (2015) (see Fig. 8). The PGA (Peak Ground Acceleration) value for artificial earthquake is 0.154 g, and the duration is 17.3 s. A procedure was applied to the bridges by incrementally increasing the earthquake amplitudes by multiplying the acceleration time history by a scalar factor. Six artificial earthquakes were 1×0.154 g, 2×0.154 g, 3×0.154 g, 4×0.154 g, 5×0.154 g, and 6×0.154 g.

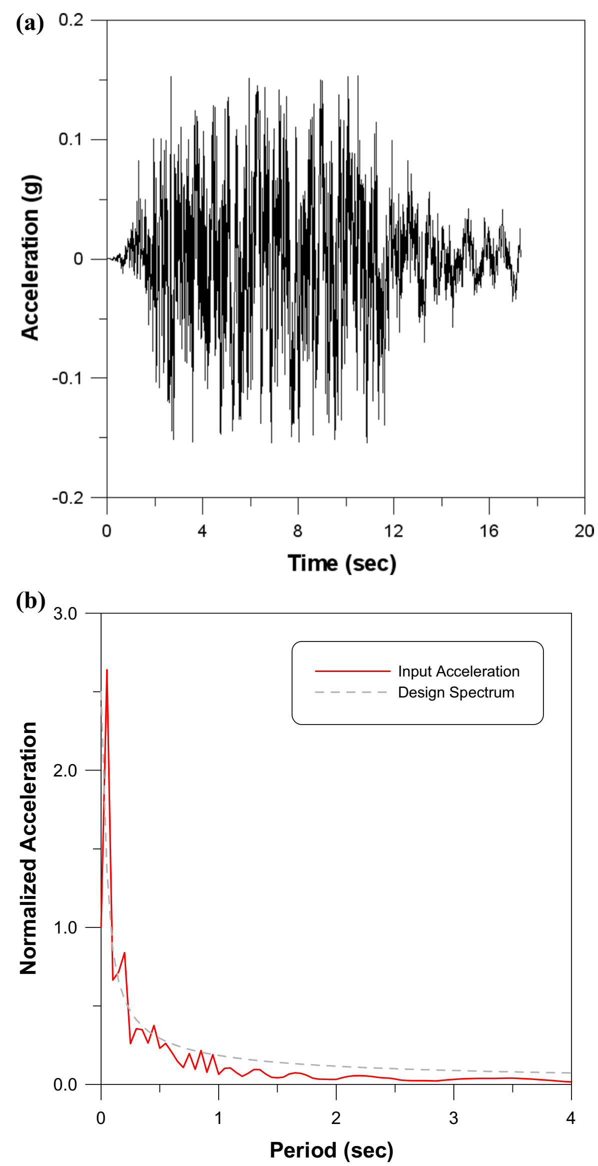

Fig. 8 Input acceleration: **a** artificial acceleration and **b** response spectrum.

The bridge slab is modelled by reinforced concrete plane stress elements in RCAHEST, as shown in Fig. 9. Fig. 9a shows the finite-element discretization and the boundary conditions for the two-span simply supported reinforced concrete bridges. Fig. 9b shows a method for transforming a circular section into rectangular strips for the purpose of using plane stress

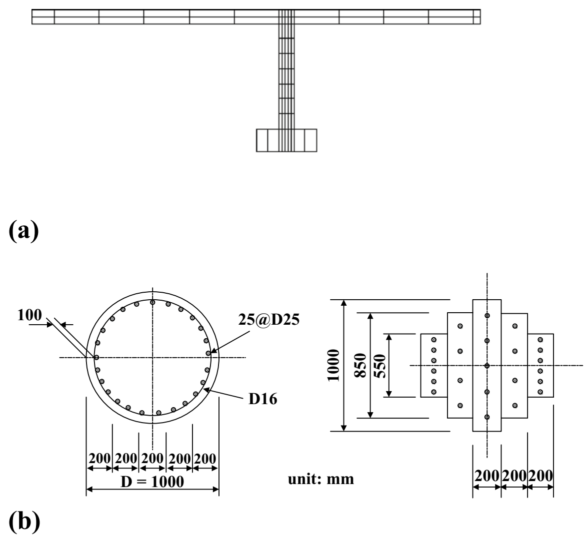

Fig. 9 Finite-element model for two-span simply supported reinforced concrete bridge: **a** finite-element mesh for two-span bridge and **b** transformation of a circular column to an idealized equivalent rectangular column.

elements. For rectangular sections, equivalent strips are calculated. After the internal forces are calculated, the equilibrium is checked. In this study, the Hilber–Hughes–Taylor (HHT) method is adopted for the solution of the dynamic equilibrium equations.

For comparison, a finite-element model of the two-span reinforced concrete bridge is also established in SAP2000 version 7.4 (Computers & Structures, Inc., 2000). The columns' plastic hinges are modeled and the confined concrete model proposed by Mander et al. (1988) is used.

The fundamental period of the two-span simply supported reinforced concrete bridge is 0.667 s. As shown in Table 2 and Fig. 10, the seismic nonlinear analysis results by SAP2000 were similar to the results by RCAHEST. The good agreement between numerical results by SAP2000 and RCAHEST demonstrates the accuracy of the proposed finite-element model.

4.2 Seismic Design Parameter Studies

This section presents on two-span simply supported reinforced concrete bridge for which additional seismic design considerations are encountered. The effects of the design parameters on the seismic responses are considered: (i) transverse reinforcement ratio, and (ii) longitudinal reinforcement ratio.

Case-1 had the same mechanical properties as original design plan in the previous section. Case-2 had the mechanical properties as Case-1, but the transverse reinforcement ratio was reduced from 0.60% to 0.40%. Case-3 had the mechanical properties as Case-1, but the longitudinal reinforcement ratio was reduced from 1.61% to 0.81%.

Case-1 showed better seismic performance than Case-2 and Case-3 (see Fig. 11). The effect of transverse and longitudinal reinforcement ratio on the seismic performance of two-span simply supported reinforced concrete bridge is large.

It is assumed that deterioration stage (Case-1D, Case-2D, and Case-3D) is completed as reduced reinforcement ratio reaches 0.36 for longitudinal and transverse reinforcing bars, respectively (see Table 1).

Analytical result comparisons between the time–displacement and time–load values for the deteriorated and reference cases are shown in Figs. 11, 12 and 13. Case-1, Case-2, and Case-3 showed better seismic performance than Case-1D, Case-2D, and Case-3D. The effect of deterioration stage on the seismic performance of two-span simply supported reinforced concrete bridge is large.

Table 2 Analytical results with SAP2000.

Item	Shear force (kN)		Bending moment (kN-m)		Axial force (kN)		Displacement (mm)	
	M	T	M	T	M	T	M	T
Abutment 1	0.0	0.0	0.0	0.0	3788.3	2873.3	40.0	33.0
Bridge column								
Upper	991.5	835.5	2985.1	1015.0	2287.9	2272.2	39.9	32.9
Lower	991.5	835.5	4130.6	3381.3	2287.9	2272.2	0.0	0.0
Abutment 2	0.0	0.0	0.0	0.0	2818.4	2084.9	40.0	33.0

M Multi-model spectrum analysis, *T* Time-history analysis.

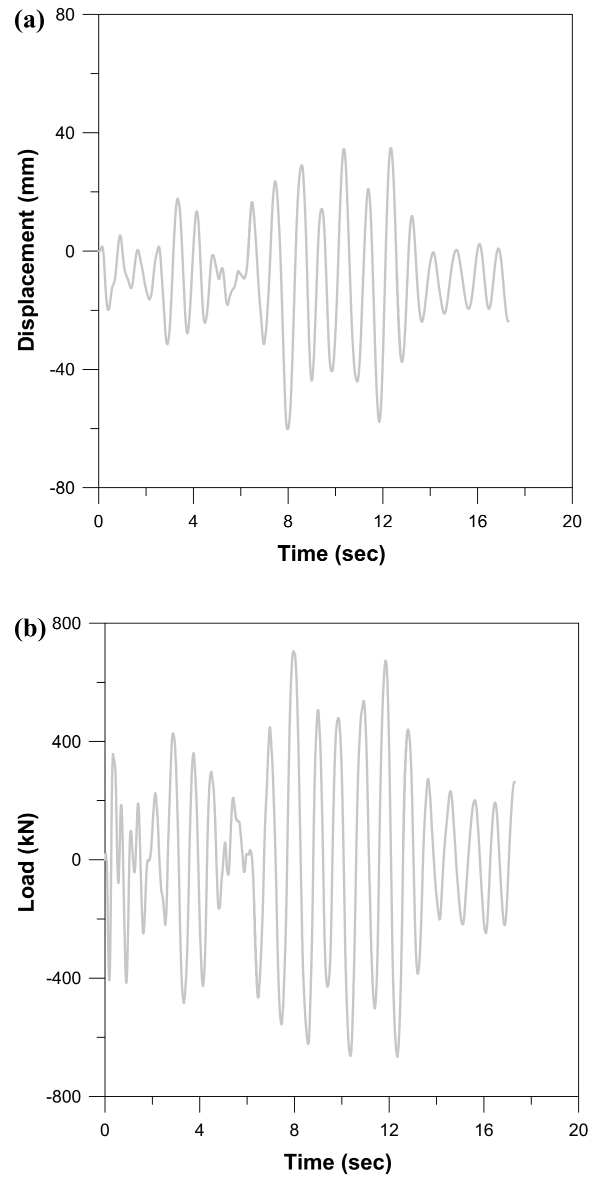

Fig. 10 Seismic response with RCAHEST: **a** displacement and **b** load.

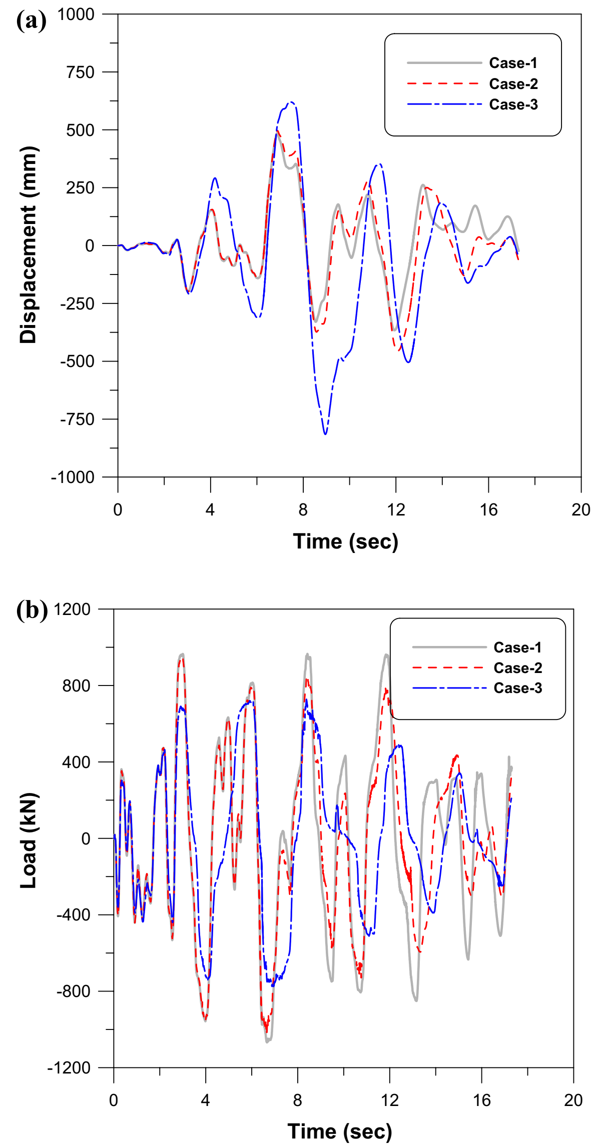

Fig. 11 Seismic response for Case-1, Case-2, Case-3: **a** displacement and **b** load.

Figs. 14 and 15 show that the Case-1 and Case-1D provided better seismic performance than the Case-2, Case-2D, Case-3, and Case-3D. Fig. 16 also shows that the Case-1, Case-2, and Case-3 provided better seismic performance than the Case-1D, Case-2D, and Case-3D. Tables 3 and 4 also show the evolution of the modified damage index, and include an assessment of physical damage incurred during numerical simulations of the earthquake loading. The damage index shows a reasonable gradual progression of damage throughout the time history of deteriorated two-span simply supported reinforced concrete bridge.

From the results of the seismic design parameter studies, the current KHBD (Korea Highway Bridge Design) design and detailing method for two-span reinforced concrete bridges in a satisfactory seismic performance for resisting seismic effects. Even after 5×0.154 g PGA earthquake damage, the bridges with the current details could be still repairable.

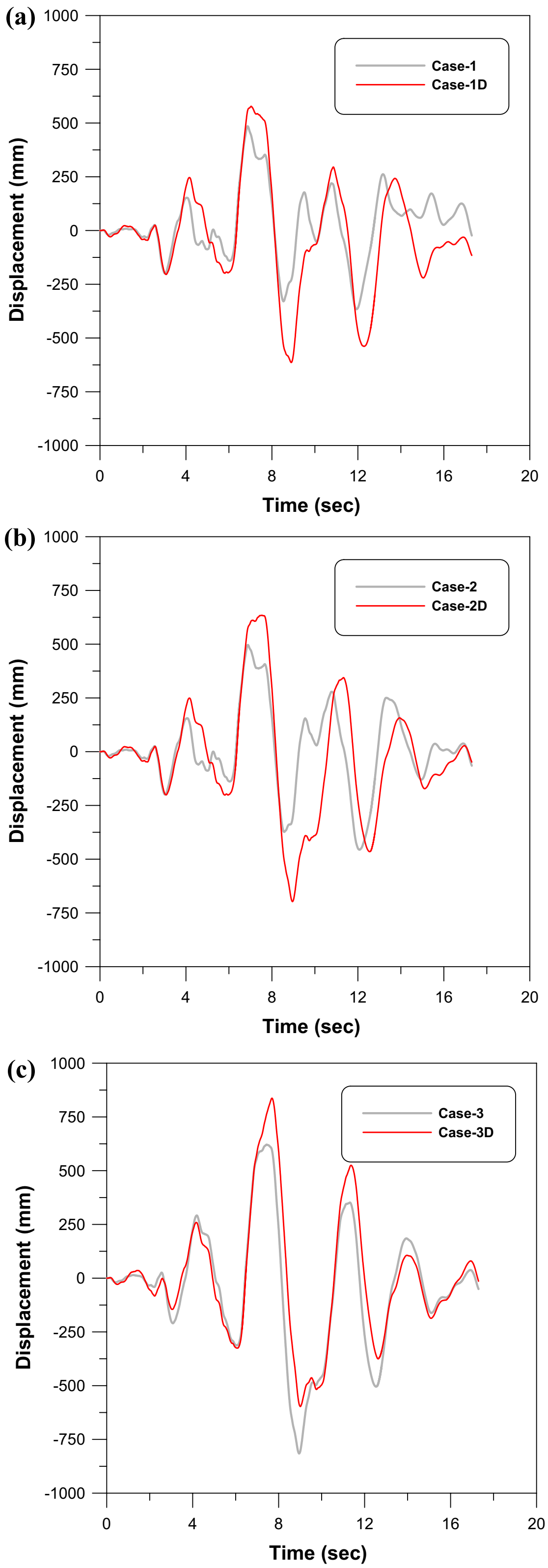

Fig. 12 Seismic responses of displacement: **a** Case-1 and Case-1D, **b** Case-2 and Case-2D and **c** Case-3 and Case-3D.

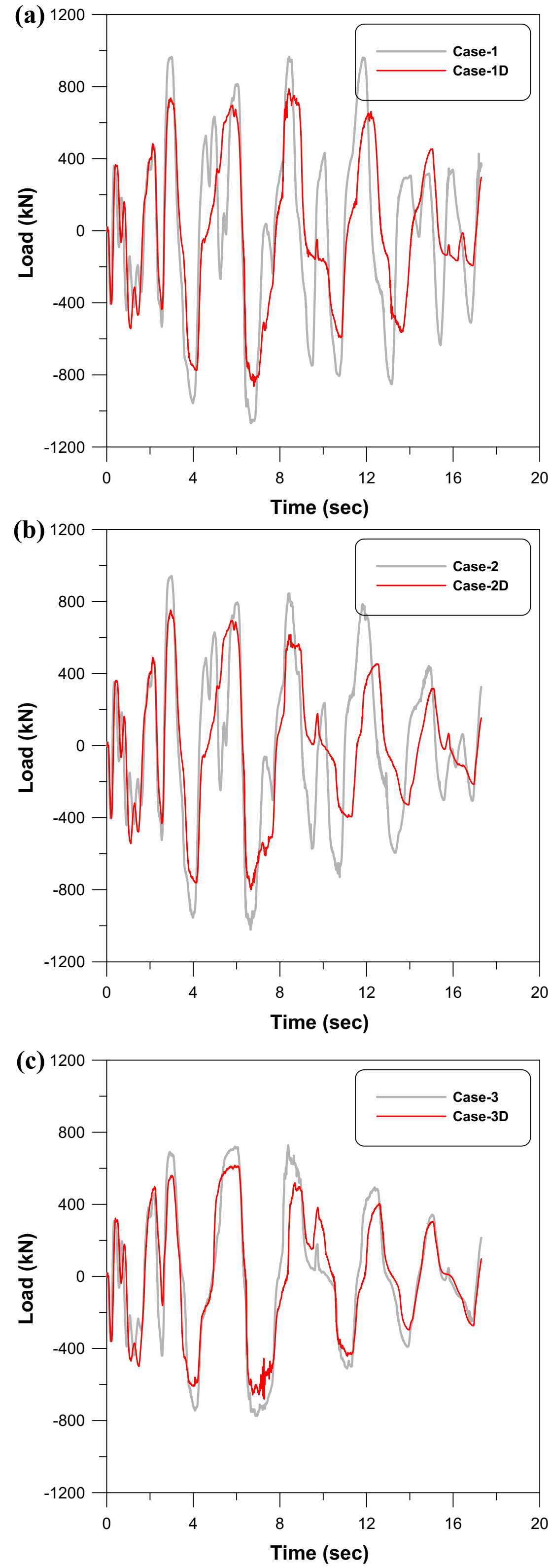

Fig. 13 Seismic responses of load: **a** Case-1 and Case-1D, **b** Case-2 and Case-2D and **c** Case-3 and Case-3D.

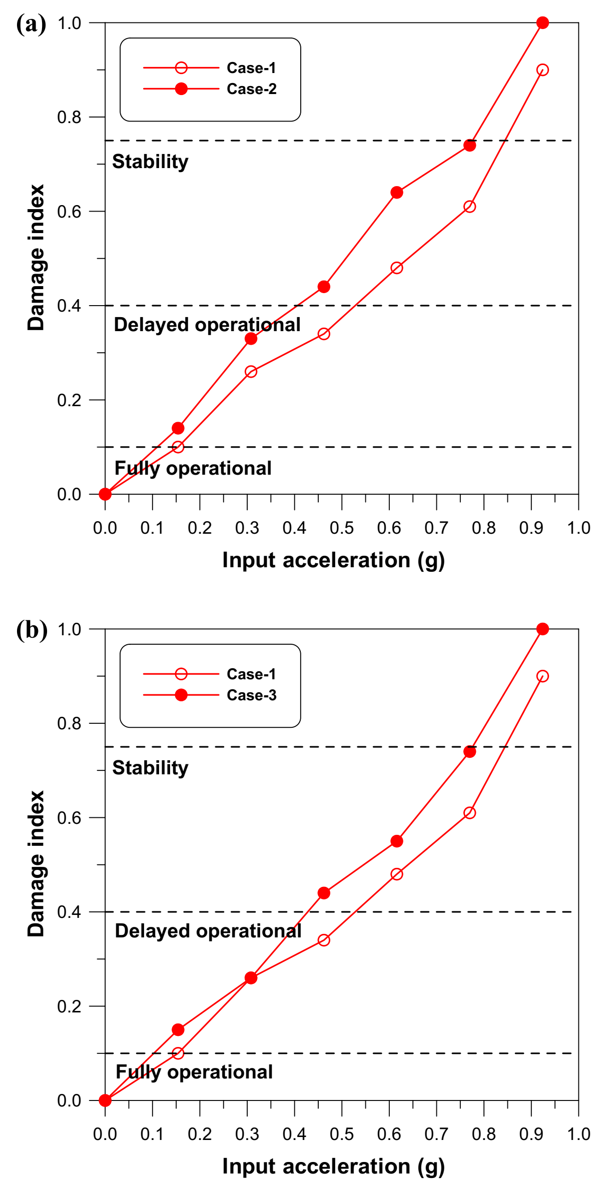

Fig. 14 Assessment of seismic performance level for cases: **a** Case-1 and Case-2 and **b** Case-1 and Case-3.

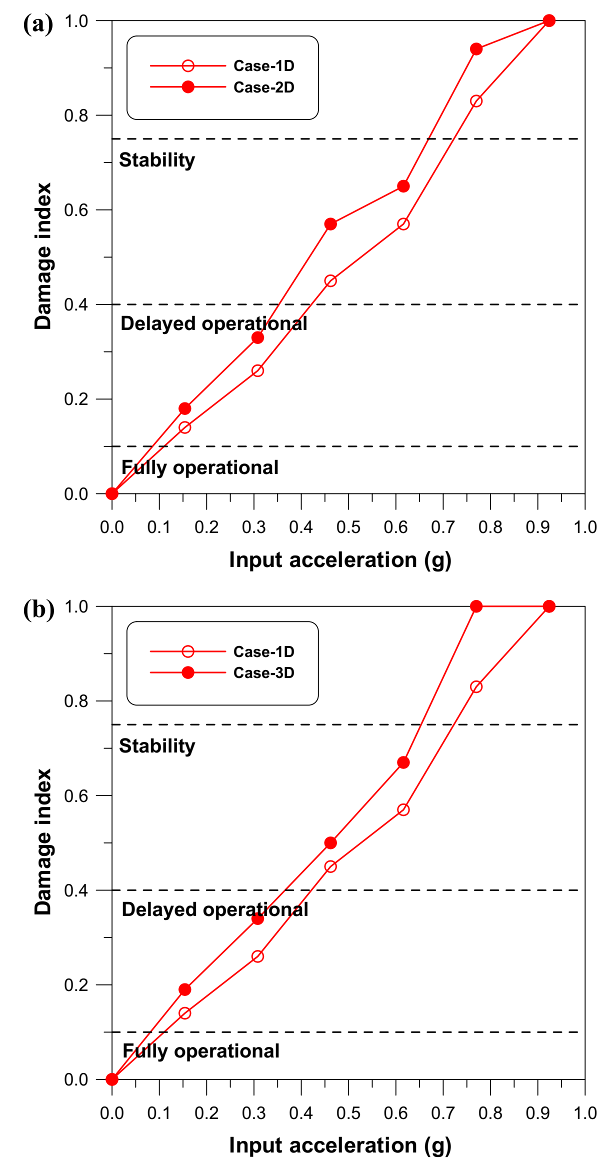

Fig. 15 Assessment of seismic performance level for cases: **a** Case-1D and Case-2D and **b** Case-1D and Case-3D.

Finally, the two-dimensional nonlinear analysis of a two-span simply supported reinforced concrete bridge under deterioration stage highlighted the effectiveness and application potentialities of the seismic performance assessment using a modified damage index.

5 Conclusions

An analytical study was conducted to quantify seismic performance of deteriorated reinforced concrete bridges using a modified damage index. From the results of the numerical analysis and evaluation of parameter studies, the following conclusions are reached.

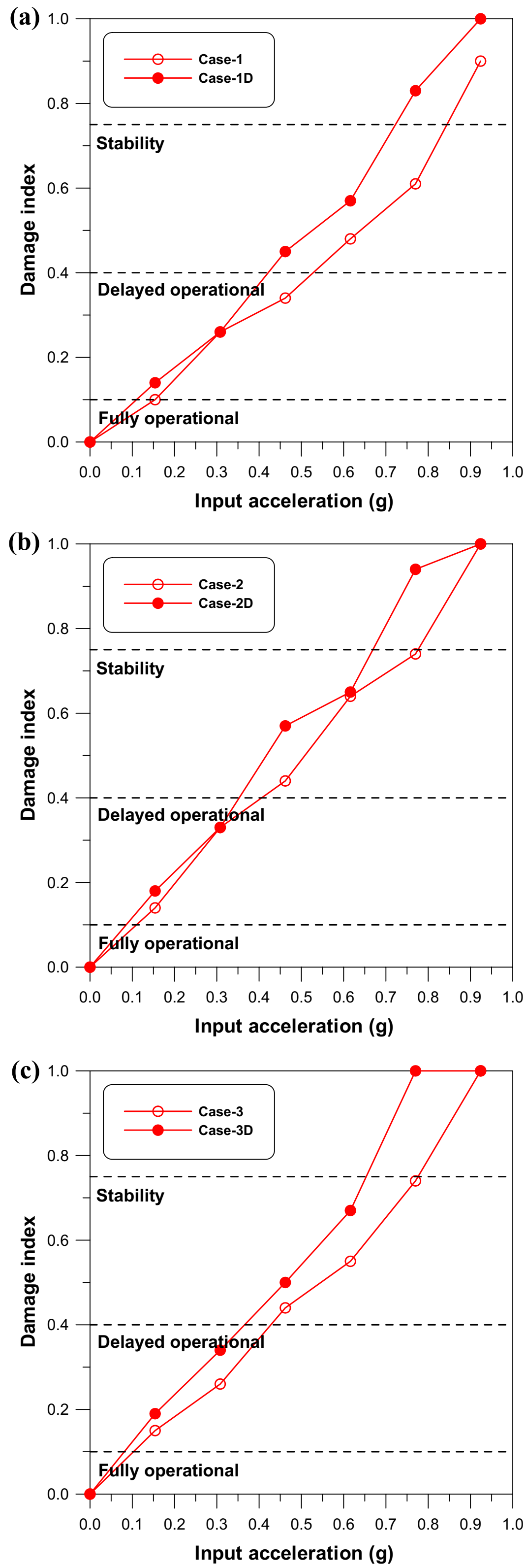

Fig. 16 Assessment of seismic performance level for cases: **a** Case-1 and Case-1D, **b** Case-2 and Case-2D and (c) Case-3 and Case-3D.

(1) A two-dimensional reinforced concrete plane stress element for nonlinear analysis of concrete structures exposed to corrosion has been presented. The proposed formulation allows to model the damage effects of uniform and pitting corrosion in terms of reduction of cross-sectional area of corroded bars, reduction of ductility of reinforcing steel, deterioration of concrete strength and spalling of concrete cover.
(2) The proposed numerical method along with results of investigation of deteriorated reinforced concrete bridges will improve the understanding of effects of deterioration on structural members. The numerical model also provides a tool that may be used to develop a better understanding of the mechanisms of damage propagation due to corrosion of the reinforcement, delamination, and spalling of reinforced concrete structures.
(3) Several parameters of deteriorated two-span simply supported reinforced concrete bridge have been studied to determine the seismic performance levels. Additional parametric research is needed to refine and confirm design details, especially for actual detailing employed in the field.
(4) Additional developments are required to integrate the effects of shear behavior, including stirrup corrosion in the damage model. These factors may be particularly related to seismic design and seismic performance assessment of deteriorated reinforced concrete bridge structures.

Table 3 Comparative evaluation of progressive damage and seismic performance level for Case-1, Case-2 and Case-3.

Input Acceleration	Case-1		Case-2		Case-3	
	Damage index	Seismic performance level	Damage index	Seismic performance level	Damage index	Seismic performance level
1 × 0.154 g	0.10	Fully operational	0.14	Delayed operational	0.15	Delayed operational
2 × 0.154 g	0.26	Delayed operational	0.33	Delayed operational	0.26	Delayed operational
3 × 0.154 g	0.34	Delayed operational	0.44	Stability	0.44	Stability
4 × 0.154 g	0.48	Stability	0.64	Stability	0.55	Stability
5 × 0.154 g	0.61	Stability	0.74	Stability	0.74	Stability
6 × 0.154 g	0.90	–	1.00	–	1.00	–

Case-1: longitudinal reinforcement ratio 1.61, transverse reinforcement ratio 0.6; Case-2: longitudinal reinforcement ratio 1.61, transverse reinforcement ratio 0.4; Case-3: longitudinal reinforcement ratio 0.81, transverse reinforcement ratio 0.6.

Table 4 Comparative evaluation of progressive damage and seismic performance level for Case-1D, Case-2D and Case-3D.

Input Acceleration	Case-1D		Case-2D		Case-3D	
	Damage index	Seismic performance level	Damage index	Seismic performance level	Damage index	Seismic performance level
1 × 0.154 g	0.14	Delayed operational	0.18	Delayed operational	0.19	Delayed operational
2 × 0.154 g	0.26	Delayed operational	0.33	Delayed operational	0.34	Delayed operational
3 × 0.154 g	0.45	Stability	0.57	Stability	0.50	Stability
4 × 0.154 g	0.57	Stability	0.65	Stability	0.67	Stability
5 × 0.154 g	0.83	–	0.94	–	1.00	–
6 × 0.154 g	1.00	–	1.00	–	1.00	–

Case-1D, Case-2D, Case-3D: longitudinal reinforcement ratio & transverse reinforcement ratio—64% of Case-1, Case-2, Case-3.

Abbreviations
A_{dx}: Area of slab; A_s: Area of the non-corroded reinforcing bar; A_{sc}: Area of the corroded reinforcing bar; C: Percentage of corrosion level; E_s: Initial bar stiffness; E_{sh}: Strain hardening rates of the bar embedded in concrete; f_{ck}: Compressive strength of concrete; f_{sy}: Steel yielding strength; f_{yh}: Yield stress of the confining steel; f_{cc}': Confined concrete compressive strength; I_y: Moment of inertia in the direction perpendicular to the bridge axis; I_z: Moment of inertia in the bridge axis direction; J: Torsional moment of inertia; R: Ratio of bond strength of corroded reinforcing bar to bond strength of non-corroded reinforcing bar; W: Mass of non-corroded reinforcing bars; ΔW: Average mass loss of corroded reinforcing bars; ε_{cs}: Compressive strain in analysis step; ε_{cu}: Ultimate strain of concrete; ε_{sm}: Steel strain at maximum tensile stress; ε_s^{av}: Average steel strain; ε_{ts}: Tensile strain in analysis step; ε_{tu}: Ultimate strain of reinforcing bars; ρ_s: Transverse confining steel ratio; σ_{sh}: Offset stress point for the initiation of strain hardening of the bar; σ_y: Yield strength of bar; σ_s^{av}: Average steel stress; Φ: Diameter; Φ_c: Fatigue parameter for concrete; Φ_r: Fatigue parameter for reinforcing bars.

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Authors' contributions
There is only one author in the current study. The author read and approved the final manuscript.

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Availability of data and materials
The research data used to support the finding of this study are described and included in the article. Furthermore, some of the data used in this study are also supported by providing references as described in the article.

Declarations

Competing interests
The author declares no competing interests.

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