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A Computational Simulation for the Damage Mechanism of Steel-Concrete Composite Slabs under High Cycle Fatigue Loads

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Abstract

This paper investigates the failure mode and the damage mechanism of steel-concrete composite slabs under high cycle fatigue loads by using three-dimensional nonlinear finite element analysis. The applicability of the simulation system, which was originally developed for reinforced concrete slabs, is extended to the steel-concrete composites with the proposed interface element, and experimentally verified with fatigue loading tests for bridge decks. The computed mid-span deflection of composite slabs shows a fair agreement with data obtained from the experiment, and the horizontally induced cracking observed in reality is properly reproduced by the computational simulation. Finally, the authors predict the ultimate state at which the upper concrete layer separated by horizontal cracks fails in compression fatigue, and the corresponding S-N diagram is computationally predicted for the future discussion.

1. Introduction

1.1 Background

There have been a lot of damaged reinforced concrete (RC) bridge deck slabs reported in Japan since 1960. To work out the causes of this problem, different groups of researchers installed a wheel-type moving load testing machine (Maeda and Matsui 1984). Various experiments of slabs have been conducted since 1980 (Matsui 1987; Pedikaris and Beim 1989). Based upon the results of these experimental studies, they could arrive at several influencing factors in fatigue problems, such as minimum thickness of slabs, and a certain requirement of rebar arrangement to be renewed in national design codes.

Alternatively, recent developments in the field of computer simulation technology have led to renewed interest in fatigue problems. Theoretical model for fatigue of plain concrete based on damage mechanics were presented (Papa *et al.* 1996; Alliche 2004). Mesoscopic analyses using Rigid Body Spring Model were also developed for fatigue problems (Matsumoto *et al.* 2008), and that showed an agreement with the characteristics of cumulative damage of concrete observed by Oh *et al.* (1991). For the simulation of structural concrete, Peerapong and Matsumoto (2006) and Maekawa *et al.* (2006a, 2006b) demonstrated fatigue simulation of RC slabs by using FE analysis.

Furthermore, there has been a growing interest to utilize steel-concrete composites for bridge decks over the past decade. The reason is due to the reduced numbers of plate girders supporting bridge decks. It is of particular importance for use in a long span situation, where the use of RC slabs is unfeasible. The fatigue perform-

ance of this newly innovated composite system has been confirmed by moving load tests (Public Works Research Institute of Japan 2001). It should be noted however that the tests were typically intended to verify the endurance limit of the composite system. No detailed investigations were conducted regarding the involved mechanism under fatigue loading as well as the ultimate strength. Therefore, a simulation system is required. This paper focuses on studying the fatigue mechanism of steel-concrete composite bridge deck slabs in high-cycle fatigue loading, and attention is directed to the characteristic cracking which is induced inside the composite slabs and not visible from outside.

1.2 Direct path-integral scheme for high cycle fatigue load

The fatigue simulation system used herein is based on the direct path integral scheme (Maekawa *et al.* 2003). The simulation is conducted by tracing the evolution of microscopic material states at each moment and location. In general, the whole hysteresis and progressive nonlinearity of consistent materials under high cycles of loading is followed up computationally with the high-speed multi-core processing technology. This finite element simulation uses three basic and essential constitutive models: compression, tension and crack shear transfer models along crack planes (**Fig. 1**).

The fatigue damage to concrete through compression and tension is taken into account by the constitutive law based on uni-axial fatigue tests (Tepfers 1979; Cornelissen *et al.* 1984). Concrete in compression not only undergoes plastic deformation but also gradually loses its elastic stiffness rooted in micro-cracks evolved in the concrete composite. This fatigue mechanism is expressed by the evolution law of the damage parameter denoted by K (El-Kashif 2004) that includes effect of loading rate (Hsu 1981). The decrease in stiffness due to fatigue in the bond between reinforcement and concrete

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	Compression model	Tension model	Crack shear model
Core Constitutive low	<p>Stress-strain</p> <p> $\sigma = E_0 K_0 \varepsilon_e$ $\varepsilon = \varepsilon_e + \varepsilon_p$ </p>	<p>Stress-strain</p> <p> $\sigma = E_0 K_T \varepsilon_e$ $\varepsilon = \varepsilon_e + \varepsilon_p$ </p>	<p>Shear stress-shear strain</p> <p> $\tau = \int_{-\pi/2}^{\pi/2} R'_c(\omega, \delta, \theta) \sin \theta d\theta$ </p>
Enhanced model for High cycle fatigue	<p>Fracture parameter K_C considers time dependent plasticity & fracturing and cyclic fatigue damage</p> <p> $dK_C = \left(\frac{\partial K_C}{\partial t} \right) dt + \left(\frac{\partial K_C}{\partial \varepsilon_e} \right) d\varepsilon_e$ </p> <p> $\left(\frac{\partial K_C}{\partial \varepsilon_e} \right) = \lambda \sim \text{when } F_k = 0$ $\left(\frac{\partial K_C}{\partial \varepsilon_e} \right) = - \left(\frac{\partial F_k}{\partial \varepsilon_e} \right) \left(\frac{\partial F_k}{\partial K} \right) + \lambda \sim \text{when } F_k = 0$ $\lambda = K^3 \cdot (1 - K^4) \cdot g \cdot R$ </p> <p>El-Kachif and Maekawa 2004</p>	<p>Fracture parameter K_T considers time dependent fracturing and cyclic fatigue damage</p> <p> $dK_T = Fdt + Gd\varepsilon_e + Hd\varepsilon_e$ </p> <p>Time dependent fracturing Cyclic fatigue damage</p> <p>Maekawa et al. 2003, Hisasue 2005</p>	<p>Accumulated path function X reduce shear associated with cyclic fatigue damage</p> <p> $\tau = X \cdot \tau_0(\delta, \omega)$ </p> <p>function original model</p> <p> $X = 1 - \frac{1}{10} \log_{10} \left\{ 1 + \left d(\delta / \omega) \right \right\} \geq 0.1$ </p> <p>Contact density model by Li & Maekawa 1989 Modification of accumulated path function by Gebreyouhannes 2006</p>
Physical meaning	Decrease of stiffness and plasticity accumulation by continuous fracturing of concrete	Decrease of tension stiffness by bond fatigue	Decrease of shear transfer normal to crack by continuous deterioration of rough crack surface

Fig. 1 Constitutive laws of concrete for high cycle fatigue analysis (Maekawa *et al.* 2006a). Fig. 2 Composition of sub-planes including non-orthogonal multi-directional cracks (Maekawa *et al.* 2003).

is reflected in the analysis as the damage parameter in progress on the tension side.

The crack plane of ordinary concrete exhibits roughness related to the size of suspended gravels. The roughness is scraped and smoothed under cyclic shear slip and the interlocking is gradually lost. In the shear transfer model stated previously, the term of damage is

added to the original formulation so as to decrease the transferred shear according to the number of cycles represented by factor X in Fig. 1.

In particular, to accelerate the computation especially for high cycle fatigue paths, the integral acceleration factor ζ is installed to the terms related to time and cumulative damage in these constitutive models

(Maekawa *et al.* 2006a). After numerous cycles, the rate of nonlinearity in progress becomes exponentially small. At this stage, the integral acceleration factor ζ magnifies the damage evolution in one step, which is equivalent to the damage caused by ζ -cycles loading. This method enables to speed up high-cycle computation.

For 3D simulation, Maekawa *et al.* (2003) has extended in-plane 2D RC models including time dependent characteristics and creep models to a 3D orthogonal space system by means of the projection-composition method (see Fig. 2). This is regarded as an extended version of the smeared crack approach (Collins and Vecchio 1982), and the composition technique is regarded as a simple extension of the multi-directional non-orthogonal fixed crack modeling. This computational framework has already been verified under low-cycle static and dynamic loads (Maekawa *et al.* 2003). Whatever the complexity of the loading hysteresis is, the multi-axial stress-carrying mechanism is formulated as a simple summation of 1D sub-mechanisms representing the cracked concrete and reinforcement.

1.3 Fatigue simulation of RC decks – Review –

The simply supported RC slab subjected to moving wheel-type loads described by Maeda and Matsui (1984) was used for verification of the computational simulation. Here, no web reinforcement was considered in both experiment and analysis. In computation, 30 N/mm^2 was assumed as the uni-axial compressive strength of concrete and the high yield strength was

assumed for reinforcement (800 N/mm^2) to prevent fatigue rupture of the steel. Analysis was carried out on a half domain of the slab with the x-coordinate (load travel direction) defined as the axis of symmetry as shown in Fig. 3 (Maekawa *et al.* 2006a).

Two types of load, cyclic fixed point loading and a traveling wheel-type one, were applied to allow for a comparison of failure mode and fatigue life as shown in Fig. 4, where the magnified displacement profile is inverted for ease of understanding. As a wheel-type load (tire) is not a concentrated sharp line but rather a belt action with breadth, forces are applied simultaneously on 6 adjacent nodal points on the central strip of finite elements, and the center of the resultant forces was gradually moved by shifting three sets of nodes on the loading FE strip. This results in a computed width of the wheel track of 173 mm .

There was a great difference in failure modes with the two types of loading. Magnified modes of failure are shown in Fig. 4. For fixed-point loading, the so-called punching shear mode of failure is identical with the conical-shaped plane of failure (Higai 1978, Kakuta and Fujita 1982, Graddy *et al.* 2002) under concentric pulsation. Localized deformation of the slab in both longitudinal and transverse directions takes place and distinct concentrated downward displacement is visible at the center of the slab. In the case of moving loads, typical shear failure modes are not in evidence in the longitudinal direction of the slab (Fig. 4). This computational simulation was consistent with experimental observations.

Fig. 3 RC bridge deck model.

Fig. 4 Magnified deformation under different types of loading (Maekawa *et al.* 2006b).

Under moving loads, multi-directional flexural cracks occur over the whole domain of the RC slab. Diagonal shear cracks in the longitudinal direction are prevented by crack-to-crack interaction (Pimanmas and Maekawa 2001). As a result, the load-carrying mechanism evolves from punching shear to semi in-plane load-carrying over the transverse direction. The three-dimensional direct path-integral scheme made it possible to reproduce the mechanistic character of failure modes and deformation.

In general, the S-N diagram is one of the most reliable methods for assessing RC bridge decks. Maekawa *et al.* (2006b) confirmed the S-N diagram for RC bridge

decks under both pulsating and moving loads by using the computational simulation (**Fig. 5**). Wheel-type loading considerably decreases the fatigue life of slabs. Roughly speaking, life is reduced by 1/100-1/1000 in the computations, consistent with experimental observations. The high cycle fatigue analysis, using the logarithmic time-integral method, was introduced as one of the specific methods that enable the prediction of fatigue lives of full-scale RC slabs (Maekawa *et al.* 2006b).

2. Extended model for Composite deck

2.1 Interface element between steel and concrete

To replicate the actual property between the steel plates and concrete, the interface element was considered (see **Fig. 6**). A state of free normal stress is assumed after debonding and separation. Afterwards, Mohr-Coulomb's law of friction is applied (Maekawa *et al.* 2008). A coefficient of friction, denoted by μ , of 0.6 on average is assumed on the surface of contact (Rabbat *et al.* 1985). In this study, the authors consider the chemical adhesion of a steel-concrete interface as the initial bonding of 2.5 MPa according to the past experiences. The rigid stiffness of open-slip modes of the interface is assumed until the tensile stress normal to the interface may exceed the initial bonding. After de-bonding, Mohr-Coulomb's

Fig. 5 S-N diagram for RC bridge deck.(Maekawa *et al.* 2006b).

Fig. 6 Interface element model.

frictional law is activated as stated above. No recovery of chemical adhesion is assumed. To simplify the model, the interface model does not take into account the change of the initially developed chemical bonding and the friction coefficient during the high-cycle fatigue in time domain.

2.2 Examination of proposed interface element

The interface element proposed above is examined against an experiment. First, a tested steel-concrete composite deck, which consists of the bottom plate, concrete slab and steel ribs as shear connectors, is shown with their material properties in **Fig. 7**. It is a 1.5 m square and the simply supported span is 1.2 m. The steel ribs are spaced 300 mm apart. The spacing of reinforcing bars is 200 mm orthogonal to the ribs. The 169 mm total thickness includes the 9 mm thick bottom steel plate. The static loading area is 200mm square at the center of slab.

The finite element discretization in space is shown in

Fig. 8. Solid isoparametric elements are used throughout the target structure. The nonlinear modeling for reinforced concrete and the plain one are described in the previous section. Reinforcing bars with surrounding concrete are modeled as a 35 mm thick reinforced concrete layer. The bottom plates and I-shape ribs are modeled as continuous elements with material properties of elasto-plasticity. All the surfaces of steel elements are covered by joint interface elements to connect with adjacent concrete. The total numbers of nodes are 12,450, and 8,400 solid elements and 2,030 joint interface ones are allocated in 3D extent.

Experimental data and simulated results are compared (see **Fig. 9**). Load versus mid-span deflection curves calculated by the analysis are almost consistent with experimental results. However, the maximum load-carrying capacity in the analysis is slightly smaller than that obtained in the experiment. The crack profile at the mid-span cross section transverse to the rib direction is also compared with contour maps illustrated in the

Fig. 7 Tested composite deck specimen.

Fig. 8 FE mesh model of composite deck specimen.

Fig. 9 Comparison experiment and analysis (Load versus mid-span deflection and crack profiles).

analysis. The photo shows two major cracks. One is the outer shear cracking that reaches via the tip of rib B and F to the root of rib A and F from the edge of loading plate, and another one is the inner shear crack that seems to be a typical punching shear crack. The contour map of 2mm mid-span deflection shows the initiation of the outer shear crack from the tip of ribs, and the contour map of 20mm mid-span deflection shows the propagation of both outer and inner shear cracks. These comparisons are thought to support the validity of the proposed interface model and the reproducibility of the finite element analysis.

3. Fatigue simulation for composite deck under moving loads

3.1 Fatigue simulation: damaging process

(1) Model profiles and procedure of moving load in analysis

Using the interface element described above, the direct

path-integral scheme, which was summarized in the previous section, is directly applied to the fatigue simulation of steel-concrete composite slabs under high cycle moving loads. The composite deck model investigated here faithfully follows the experimental study reported by Sakurai *et al.* (2005) (see Fig. 10). It consists of the bottom steel plate, I-shape steel, reinforcing bars and concrete. The plane dimensions of the slab are 2.4 m x 4.5 m and 166 mm thick including a 6 mm thick bottom plate. I-shape steels in transverse direction are set etc 200 mm. Single layers of reinforcement consisting of D16 etc 200 mm are provided on the I-shape ribs in the longitudinal direction. Because of its symmetry, only half of the slab is modeled in the analysis (see Fig. 10). Solid elements are used throughout for the model in which material property is presented in Table 1. Since the bottom plate and I-shape ribs were welded, they are modeled as continuous elements but with different material properties accordingly. All the surface of steel element is covered by interface element. The total num-

Table 1 Material property of composite deck.

	Strength N/mm ²	Stiffness N/mm ²	Poisson's ratio
Concrete	Compressive $f_c = 36$	3.0×10^4	0.17
	Tension $f_t = 2.5$		
Reinforcement bar (D16)	Yield point $f_y = 400$	2.0×10^5	0.30
Bottom plate	Yield point $f_y = 350$	2.0×10^5	0.30
I-shape rib	Yield point $f_y = 400$	2.0×10^5	0.30

bers of nodes are 18,580, and 11,088 solid and 4,158 interface elements are arranged for the 3D analysis domain.

The moving point load rolls back and forth in a range of 3.0 m, as is shown in **Fig. 10**, in both experiment and analysis. The travelling load pattern is produced in the analysis by applying linearly varying nodal forces in each load step with phase-shift along the wheel running line. As a result, the total nodal force is always kept constant during the passage of moving loads but the gravity center of the applied nodal forces moves step-

by-step. The wheel load increases in increments of 19.6 kN every 40,000 passages from 157 kN to 392 kN so as to match the experiment conditions. Beyond 520,000 passages, fatigue analysis continues under the maximum loading of 392 kN. **Figure 11** shows the step-wise incremental loading pattern versus the number of passages used in this analysis and experiment.

To obtain the progress of maximum and minimum mid-span deflections in experiments, the static load is applied at the center of the slab every 40,000 passages of the wheel loading. The minimum deflection means the residual one after complete unloading. The loading speed was also set forth consistently with the experimental condition. Then, the inertial forces and impact can be automatically taken into account in the simulation. In fact, the dynamic responses with inertia forces are generally different from the quasi-static responses in reality. It has been reported that the dynamic responses obtained in the past were about 10% larger than the static values (JSCE, 2008).

(2) Progress of mid-span deflection and chemical bonding

Figure 12 shows the progressing deflection at the span center and cyclic numbers from the analysis with different chemical bonding, together with experimental data

Fig. 10 The target composite deck model.

by Sakurai *et al.* (2005). The overall trend is simulated successfully. The maximum deflection in the analysis with chemical bond of 2.5 MPa is slightly underestimated (Fig. 12(a)). In contrast, the calculated maximum deflection assuming no chemical bond is larger than that of the experiment, and the residual deflection in the analysis is slightly higher than the experimental results up to about 200,000 passages (Fig. 12(b)). According to the material experiments and experiences in practice, the chemical bond strength ranges very widely (from zero to the level of tensile strength of concrete) and is much influenced by the local bleeding and consolidation of fresh concrete. In this experiment, 1.0 MPa seems to be close to the initial bond strength (Fig. 12(c)). Although it is hard to explicitly identify the realized chemical bonding strength inside the structure, it may be within the reasonable range according to the past experimental results.

(3) Characteristic cracking from the tip of ribs

A photo of the test specimen's cross section after the experiment is shown in Fig. 13. The horizontal cracks starting from the tip of steel ribs can be observed. To compare this failure mode with the analysis with the chemical bond of 1.0 MPa (Fig. 12(c)), the contour maps of vertical strains in the cross section (Y-Z section) at the center of the span at 320,000 passages and 520,000 passages are shown in Fig. 14. The vertical tensile strains of about 1,000 μ or larger on average are concentrated in the concrete layer surrounding the upper flanges of the ribs, and the tensile strains extend horizontally (see Fig. 14(a)). This leads to the occurrence of cracking in the horizontal direction and the propagation of cracks between the upper flanges of the ribs (see Fig. 14(b)); that is to say, the damage process observed in the experiment is reasonably reproduced by the analysis. This failure mode is common to all cases with different chemical bonding.

To further analyze the mechanism by which cracks form and propagate in the horizontal direction, attention is focused on an element between the ribs at the center of the span (see Fig. 14). Calculated stress and strain of the referential element are shown in Fig. 15. Figure 15(a) indicates the tensile stress of the referential element in the vertical (Z) direction with the number of passages. Vertical tensile stress actively responds and increases its amplitude up to 240,000 passages (255 kN) and then stop increasing the positive stress thereafter, because the vertical stress of the referential element reaches the tensile strength of the concrete and cracks start at about 240,000 passages. Moreover, the strain of the referential element in the vertical (Z) direction rapidly increases after 240,000 passages (Fig. 15(b)). This supports the finding that cracks start to open in vertical direction after this moment.

Figure 15(c) shows shear stress in the X-Y plane in the referential element. The shear stress seems to decrease its amplitude after 360,000 passages. In contrast,

Fig. 11 Incremental loading pattern versus number of cycles.

(a) With chemical bond of 2.5 MPa

(b) Without any chemical bond

(c) With chemical bond of 1.0 MPa

Fig. 12 Progress of deflection at center of the deck with number of cycles.

Fig. 13 Cracks observed after test (Sakurai *et al.* 2005).

Fig. 14 Contour map of vertical strain.

shear strain gradually increases after about 400,000 passages (314 kN) (**Fig. 15(d)**). This indicates that the horizontal cracking occurs at 240,000 passages, although it does not rapidly propagate to other elements. Each horizontal crack gradually spreads and connects with other adjacent cracks up to 360,000 passages. Even at this moment, however, the slab sustains load and the mid-span deflection does not obviously increase (see **Fig. 12(c)**). The obvious increase of mid-span deflection can be seen after 400,000 passages (see **Fig. 12(c)**). The possible explanation is that even if horizontal cracks become partially connected, the integrity of the slab is not suddenly lost because of the interlocking effect at the cracked surface of concrete. The whole damage process indicated in the analysis seems to be reasonable.

3.2 Fatigue life prediction: predicted ultimate limit state

Once the number of passages reaches 520,000, at which

the experiment was over, the analysis continues under constant moving wheel loading of 392 kN. The simulation reproduces the moving load continuously applied until failure without unloading process. **Figure 16** shows the dynamic deflection response, which is simulated by assumption of a chemical bond of 1.0 MPa, with the number of passages. The analysis predicts that deflection increases monotonously on a logarithmic scale after the 520,000-passage point under the constant load of 392 kN.

Figure 17 shows the overall deformation of the slab at the maximum dynamic deflection of about 7 mm and 10,000,000 passages. Although there is no observed punching shear failure, localized deformation is noted in concrete near the zone where the moving wheel load acts. The analysis infers the following damage mechanism (scenario): cracks are formed from the tip of the ribs and these connect over the whole slab after cyclic loading, dividing the concrete slab into two layers; in

Fig. 15 Changes in stress and strain in the element under focus with number of cycles.

the ultimate state, the concrete in the upper layer undergoes compression fatigue failure and the entire slab loses stiffness. The failure mode predicted in the analysis is the compression fatigue failure instead of punching shear failure or yielding of steel. The tensile stress in the bottom steel plate is 212 N/mm^2 , and the steel does not yield at this moment.

For further understanding, the numerical approach is extended to the fatigue life assessment of steel-concrete composite slabs in terms of S-N diagrams. By considering the past criterion in practice, the amplitude of fatigue loading should be classified by the static load-carrying capacity. Thus, the static analysis is carried out as a preparation for S-N diagrams. The finite element analysis model is the same as shown in Fig. 10, and the chemical bond in this static analysis is 1.0 MPa on average. The area on which the static loading is applied is 200 mm wide in the longitudinal direction and 500 mm long in the span direction of the slab (200 mm by 250

Fig. 16 Change in deflection at center of the deck with number of cycles (dynamic response).

Fig. 17 Deformation at 10,000,000 cycles (displacement scaled up two times).

mm in the half model). The static force is applied by means of the forced nodal displacement control, and the pulsating load is produced by linearly varying nodal force in the analysis. The load versus deflection curve at the center of the slab is shown in Fig. 18. The computed maximum load-carrying capacity and the deflection at this maximum loading are about $1,514 \text{ kN}$ and 19.4 mm , respectively. As a result of this preliminary analysis, the fatigue life is hereafter defined as the number of passages at which the center deflection of the slab reaches the one corresponding to the static capacity as 19.4 mm . This definition is the same as that applied for the fatigue experiments of slabs.

The S-N diagram of the slab under the fixed-point cyclic loading and the moving wheel cyclic loading are shown in Fig. 19. The location and area of fixed-point loading are the same as the static analysis. The vertical axis is the amplitude normalized by the computational static capacity. The minimum load was set as zero. The blue line in the figure represents the formula of the fatigue life of slabs given by the Standard Specifications of Concrete Structures of the Japan Society of Civil Engineers (JSCE, 2008), assuming that the bottom steel plate forms part of the effective quantity of reinforce-

Fig. 18 Load-deflection at center of the deck (static analysis).

Fig. 19 Evaluation of composite decks under fixed-point loading and moving wheel loading.

ment in RC slabs. The arrow on the figure means that the specimen survives more than the plotted number of cycles.

Under fixed-point loading, the fatigue life at the higher amplitude agrees with the line given by the formula. This implies that the failure mode of the composite deck at high amplitudes is the same as that of RC slabs. In contrast, the fatigue lives at the lower amplitude tend to be longer than that given by the JSCE formula. This implies that the failure mode of the composite deck at low amplitudes might be different from the punching shear, which comes into sight at relatively high amplitudes.

All the fatigue lives investigated in this study under moving loading are significantly shorter than that under the fixed-point loading. This was expected based on the past experimental and analytical studies on RC slabs (Matsui 1987; Pedikaris and Beim 1989). But, four or five order difference was much larger than the past experiences. As described before, the mechanism that shortens the fatigue life of the composite slab under heavy moving loading is the separation of concrete into two layers as a result of horizontal cracking. The separation caused

by the horizontal cracks leads to the fatigue failure of the upper layer concrete. This indicates that the failure mode affects the fatigue life. Moreover, the failure mode of composite structures is greatly influenced by the mechanical property of shear connectors (Hawkins and Mitchell, 1984), because shear connectors can be a cause of cracking inside concrete. Further experimental and analytical investigations are needed to examine this issue.

4. Conclusions

1. RC and steel-concrete composite bridge deck slabs are simulated with full 3D nonlinear finite element analysis.
2. For the analysis of a steel-concrete composite structure, the interface element between concrete and steel members is defined based on Coulomb's friction law with chemical adhesion as an initial bonding strength.
3. The interface element proposed above is examined against an experiment. The comparisons of experiment and simulation are thought to support the validity of the proposed interface model and the reproducibility of the finite element analysis.
4. The progress of mid-span deflection under a high cycle moving load is successfully reproduced by analysis with the proposed interface element. Furthermore, horizontal cracks starting from the tip of I-shape steel can also be successfully reproduced in the simulation.
5. The analysis indicates that even after horizontal cracks formed, the slab can still partly sustain a moving load because of the inter-locking of the rough crack surface. This mechanism is gradually lost due to repetitive moving loads. As a result, concrete is separated into two layers.
6. More importantly, the ultimate state of the composite slab, where the upper layer concrete fails in compression fatigue, is predicted by analysis.
7. The S-N diagram of composite slab is shown based upon numerical analyses in this study.

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