A PROPOSAL OF DURABILITY ASSESSMENT FOR EXISTING BRIDGE DECKS SUBJECTED TO HEAVY TRAFFIC

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***ABSTRACT*:** A damage progress of RC slab which had been obtained in past experiments under the high cycle moving load was simulated by using nonlinear finite element analysis. The progress of deflection and the strain of reinforcing bar show good agreement with past experiments. The fatigue life can be also reasonably predicted. Furthermore, the effect of 70mm thick asphalt pavement was also examined in the simulation. Based on this analysis, three simple indexes; normalized mid-span deflection, increment of curvature and growth rate of acceleration at the center of slab, were proposed to know the damage state of existing bridge deck. The applicability of the indexes is also discussed from the practical view point. Consequently, the possibility to combine the concept of the indexes to the current measuring method is shown.

KEYWORDS: existing bridge deck, life cycle assessment, fatigue

1. INTRODUCTION

The assessment of deck slabs has been one of the criteria to manage bridges. In general, slabs need repairing earlier than main structures since they directly sustain heavy traffic loads. In Japan serious damages of RC slabs have been reported since 1960.

To work out from that the problem had been considered, different groups of researchers installed a wheel-type moving load testing machine (Maeda and Matsui 1984). Various experiments have been conducted since 1980 (Matsui 1987; Pedikaris and Beim 1989). Based upon the results of these experimental studies, they could come up into several influencing factors in fatigue problems. Such as the minimum thickness of slab and a certain requirement of rebar arrangement were renewed in design codes.

Here, one question was raised on existing bridge decks that were not designed based on the renewed code. It is possible to say that the existing bridge decks are exposed to higher risk than recent ones. To assess the damage of existing slabs, several analytical studies have done (Maekawa *et al.* 2006a, b; Fujiyama *et al.* 2008), but further progress is required for practical use.

This paper focuses on assessment for existing bridge decks. The measurable data is quite limited on site while numerical simulation gives us variable information. Thus, the authors try to identify the damage of real structures by using a simple index based on both past experimental fact and the simulation results in this study.

2. ANALYTICAL STUDY

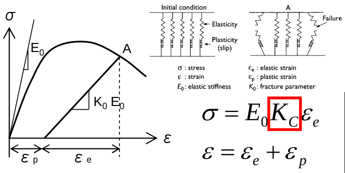
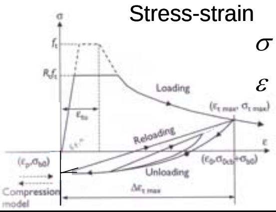
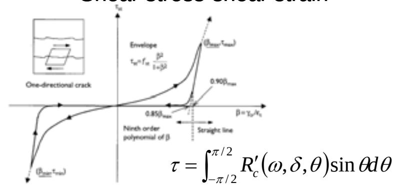
	Compression model	Tension model	Crack shear model
Core Constitutive laws	Stress-strain  $\sigma = E_0 \boxed{K_C} \varepsilon_e$ $\varepsilon = \varepsilon_e + \varepsilon_p$	Stress-strain $\sigma = E_0 \boxed{K_T} \varepsilon_e$ $\varepsilon = \varepsilon_e + \varepsilon_p$ 	Shear stress-shear strain  $\tau = \int_{-\pi/2}^{\pi/2} R_c'(\omega,\delta,\theta)\sin\theta d\theta$
Enhanced model for High cycle fatigue	Fracture parameter Kc considers time dependent plasticity & fracturing and cyclic fatigue damage $\boxed{dK_C} = \underbrace{\left(\frac{\partial K_C}{\partial t}\right)dt}_{\text{time dependent}} + \underbrace{\boxed{\left(\frac{\partial K_C}{\partial \varepsilon_e}\right)d\varepsilon_e}}_{\text{cyclic fatigue}}$ $\left(\frac{\partial K_c}{\partial \varepsilon_e}\right) = \lambda \sim when F_k = 0$ $\left(\frac{\partial K_c}{\partial \varepsilon_e}\right) = -\left(\frac{\partial F_k}{\partial \varepsilon_e}\right)\left(\frac{\partial F_k}{\partial K}\right) + \lambda \sim when F_k = 0$ $\boxed{\lambda = K^3 \cdot (1-K^4) \cdot g \cdot R}$ El-Kachif and Maekawa 2004	Fracture parameter K_T considers time dependent fracturing and cyclic fatigue damage $dK_T = \underbrace{Fdt}_{\text{Time dependent fracturing}} + \underbrace{\boxed{Gd\varepsilon_e}}_{\text{Cyclic fatigue damage}} + Hd\varepsilon_e$ Maekawa et al. 2003, Hisasue 2005	Accumulated path function X reduce shear associated with cyclic fatigue damage $\tau = \underbrace{\boxed{X}}_{\text{function}} \cdot \underbrace{\tau_0(\delta,\omega)}_{\text{original model}}$ $\boxed{X} = 1 - \frac{1}{10}\log_{10}\left\{1 + \int \lvert d(\delta/\omega) \rvert\right\} \geq 0.1$ Contact density model by Li & Maekawa 1989 Modification of accumulated path function by Gebreyouhannes 2006
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

Figure 1. Constitutive laws of concrete for high cycle fatigue analysis (Maekawa et al. 2006a)

2.1 Nonlinear FE analysis for fatigue

The fatigue simulation system used herein is based on the direct path integral scheme (Maekawa *et al.* 2003). The finite element package used here three basic yet essential models that are; compression, tension and crack shear models. This simulation is conducted by tracing the evolution of microscopic material states at each moment. These three are important to treat the cumulative fatigue damage and time dependency effects (Figure 1.).

For 3D simulation purpose, Maekawa *et al.* has extended in-plane 2D RC models including time dependent models and creep models to 3D orthogonal space system by means of the composition method (2003). This composition technique is regarded as a simple extension of the multi-directional non-orthogonal fixed crack approach.

The simulation system has been enhanced for high cycle fatigue analyses by using a logarithmic integral method (Maekawa *et al.* 2006a). The fatigue mechanism of RC slab subjected to wheel type moving load was successfully simulated by this simulation (Maekawa *et al.* 2006b).

2.2 Model

The RC slab investigated here is based on experiments reported by a sub committee of JSCE; Investigation for Bridge Deck Slab (2008). It managed the moving load tests with six organizations at the same time. In this project, three specimens were designed as exactly the same dimensions. The dimensions of the slab are 2.8 m x 4.5 m and 190 mm of thickness. The arrangement of reinforcing bar is shown in Table 1. The authors adopted this specimen as a prototype to examine the simulation results.

In the experiments, slabs are directly supported in longitudinal direction and another side is elastically supported by steel girders that are sustained by main frame. In the simulation, direct support is provided in longitudinal direction and no

Table 1. Arrangement of reinforcing bar

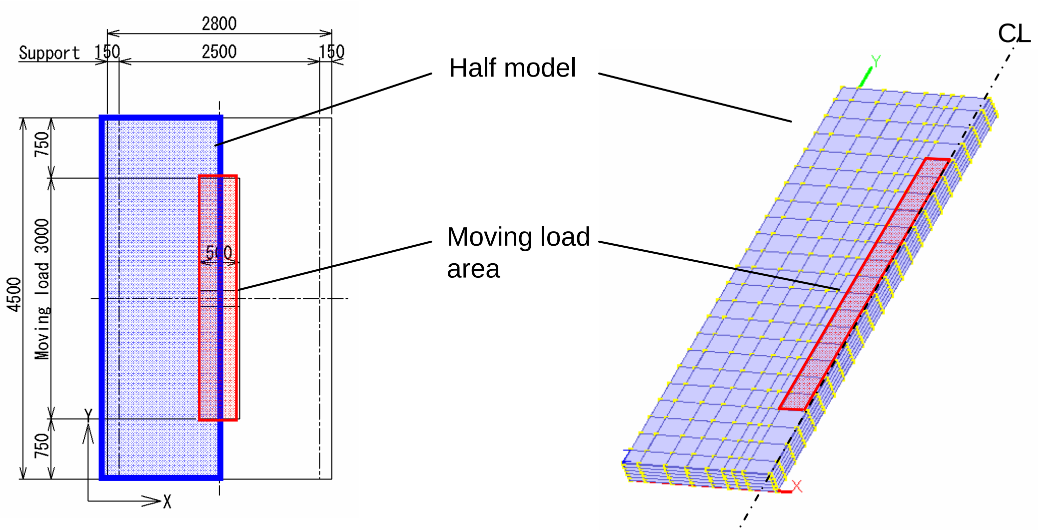
	Transverse (X)	Longitudinal (Y)
Compression side (cover 30mm)	D16ctc300	D13ctc300
Tension side (cover 30mm)	D16ctc150	D10ctc300

Yield point (N/mm^2)	345
Strength (N/mm^2)	500
Young's modulus (N/mm^2)	200,000

Table 2. Material property of concrete and pavement

N/mm^2	Specimen No.1*	Specimen No.2*	Specimen No.3*	Concrete	Pavement
Compressive strength	30.7	28.8	31.9	30	---
Young's modulus	28,200	26,300	28,000	28,000	2,000

* taken from *Required performance and maintenance technology for bridge deck*, sub Committee of JSCE, 2008.



(a) Dimension of specimen

(b) Model for analysis

Figure 2. RC slab model in this study

restraints are given in transverse direction. A half of the slab is modeled for simulation simplicity because of its symmetry (Figure 2). The property of concrete is decided based on an average of three specimens as shown in Table 2. The constant round trip load 160 kN, 3.0m/s is applied as same as experimental condition.

The effect of pavement thickness of 70 mm thick pavement is also studied as an extra model. The pavement is assumed that it consists of mainly asphalt. To simplify, it is modeled as an elastic material. In order to provide a good serviceability, the pavement is carefully maintained with a certain period in the reality. In other words, it is hardly possible that pavement lose its stiffness totally. Therefore, it might be reasonable that fatigue failure of the pavement is neglected in this study. The property of the pavement is also provided in Table 2. It is widely known that asphalt is one of the strongly temperature-dependent materials. Nishizawa and Kobayashi calculated the stiffness of 75 mm thick asphalt-based pavement on bridge deck as 1,990 N/mm^2 (for average temperature of 28.5℃)～9,566 N/mm^2 (for average temperature of 4.5℃) (2004). In this simulation, a unique stiffness of 2,000 N/mm^2 is given for the pavement. Poisson ratio is a constant value about 0.35.

2.3 Results

Figure 3 shows the progress of maximum mid-span deflection by live load. The simulation results of

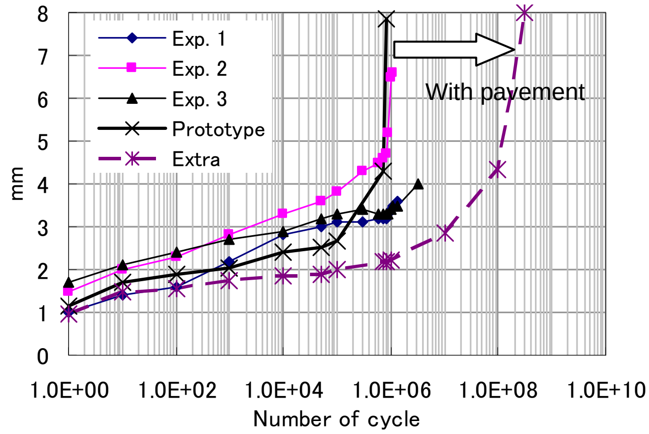


Figure 3. Progress of mid-span deflection

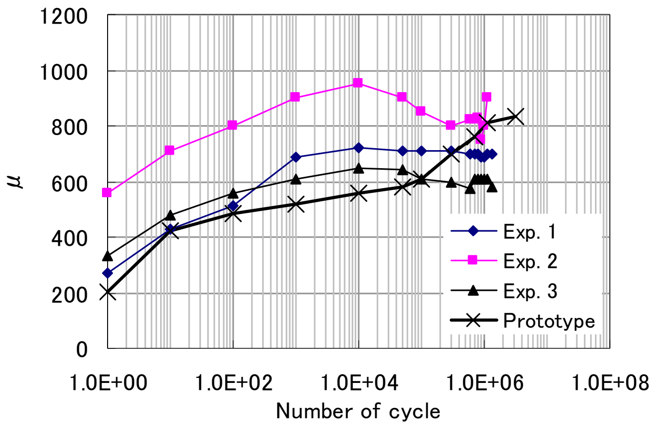
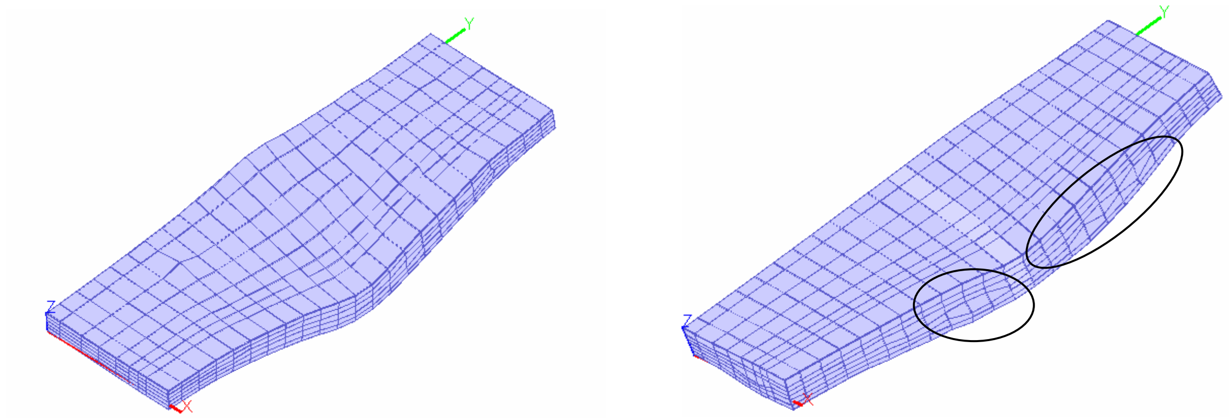


Figure 4. Strain of main rebar (bottom, dir-X)



(a) Prototype at 800,000 cycles

(b) Extra model at 30,000,000 cycles

Figure 5. Deformation at 10mm deflection (disp. x 30)

prototype exist in the band of experimental scatter during first 1,000 cycles. While it shows lower value approximately during 1,000-100,000 cycles, the deflection grows rapidly and fails at around 800,000 cycles. According to the report (sub committee of JSCE 2008), the specimen No.1 and No.2 failed at 1,342,300 cycles and at 1,066,000 cycles relatively, however the specimen No.3 survived until 3,250,000 cycles. It means that the fatigue life of specimen No.3 is longer than 3,250,000 cycles. The computed fatigue life can be predicted in the band of experimental scatter. The simulation is thought to be reasonable.

The progress of main rebar strain at the mid-span is shown in Figure 4. All the strains show steady increment until around 10,000 cycles, although the experiment No.2 is obviously higher than others. The trend subsequent to 10,000 cycles is associated with the progress of mid-span deflection except the case of experiment No.2. The possible explanation for experiment No.2 is that the strain gage is located just close to the major crack. In that case, the gage can show higher value at early stage and it decreases later because of re-structured stress distribution by other cracks. As a whole, the stress calculated from the strain is at most 160 N/mm^2 at around 1,000,000 cycles. It is almost a half of yield strength.

Another concern of this study is the effect of pavement. Interestingly, the deflection of extra model is illustrated roughly 10~15% smaller than that of prototype until around 100,000 cycles at which the deflection of prototype starts to rapid growth. The fatigue life is also more than 100 times loner than that of the prototype. A 70 mm thick pavement can be a positive effect to the life of slab. At this point, it is suggested that the design which do not consider the stiffness of the pavement might be conservative.

On the contrary, there are negative effects to the fatigue life caused by water, shrinkage, poor construction skill and so on. Thus, it is considered that the positive effects and negative effects might cancel each other in reality.

The deformations at 10mm deflection are presented in Figure 5. For the case of extra model, some localized deformations are observed (Figure 5(b)). It means that even if the pavement which is carefully maintained increases the fatigue life of slab, the damage is accumulated to the concrete.

3. PROPOSAL OF DAMAGE INDEX

3.1 Normalized mid-span deflection

To generalize the results shown in chapter 2, the deflection d is normalized by the value of the deflection d_0 obtained at the first cycle (Figure 6). There seems to be a correlation between the value d/d_0 and the fatigue life of slabs. The specimen No.1 and No.2 failed where the d/d_0 marked higher than 3.5 as well as the simulation. To take into account a safety factor, d/d_0=3.0 is recommended as a damage index for practical use.

3.2 Curvature

The curvature is also possible to be one of the useful indexes to recognize the damage progress before sudden shear failure. The report (sub committee of JSCE 2008) defined the curvature as equation (1) and illustrated in Figure 7.

$$1/r_x = (W_2-2W_3+W_4) \quad / \lambda^2 \cdot\cdot\cdot\cdot\cdot\cdot (1)$$

Here, $1/r_x$; curvature, λ ; a certain distance from center line (=600mm), W_2, W_4 ; deflection at λ, W_3 ; deflection at center line.

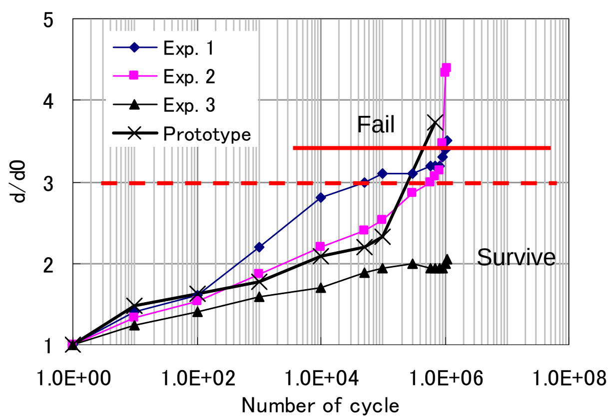


Figure 6. Normalized mid-span deflection

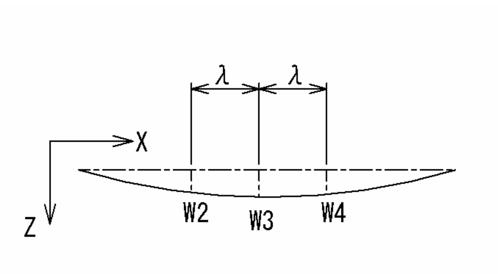


Figure 7. Definition of curvature in this study

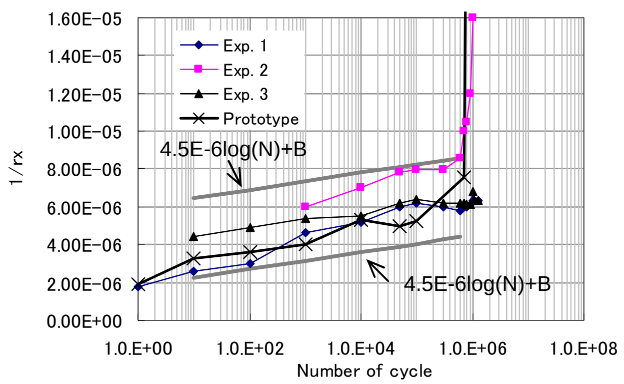


Figure 8. Progress of curvature

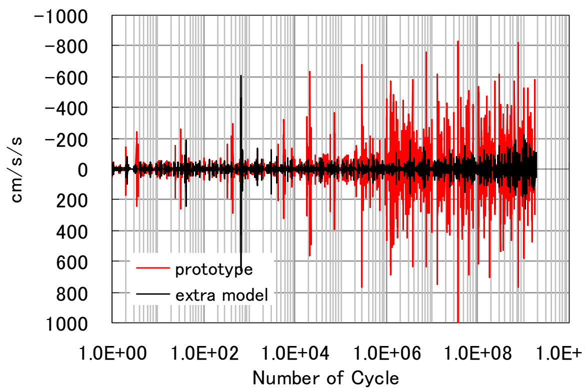


Figure 9. Growth of acceleration-Z at mid-span

Figure 8 depicts the curvature versus number of cycle. The rate of increment seems stable until just before failure of Specimen No.2. It indicates that the change of the increment rate let us detect the danger. The constant increment rate in this study is shown in Figure 8 as a thick solid line that is described by equation (2) as follows;

$$1/r_x = 4.5\text{E-}6 \log(N) + B \cdot\cdot\cdot\cdot\cdot\cdot (2)$$

$1/r_x$ is the index of curvature. The valuable N means the number of cycle. B is a particular value for each case.

3.3 Acceleration

The measurement of acceleration is the method that is widely known as a useful tool to evaluate structural damage (Nagayama *et al.* 2005). Although an available data does not exist in the experiments, the authors illustrated the vertical acceleration at the center of the slab which is obtained from the simulation (Figure 9). It seems that at the point where the acceleration starts to grow correlated to the progress of mid-span deflection. Thus, the acceleration may be possible to be one of the damage indexes, even though a lot of dispersion is perceived in the figure.

4. DISCUSSION

The proposed three indexes should be considered their applicability from the practical view point.

First, to calculate the normalized deflection, the deflection at first loading are needed. For existing bridge deck, it is not easy to find it out. In that case, the simulation can substitute for it.

Second, the method of measuring displacement is required for the index $d/\ d_0$ and $1/r_x$. Especially for the curvature, accurate measurement is indispensable. This means that an unmovable point is needed as a standard to obtain the absolute value of displacement. The main frame can not be a standard because of the vibration by traffic load. Even the ground level may not be a standard since it can be changed for a long term. In other words, the direct measurement of displacement is not realistic for existing bridge deck. This problem should be solved.

Third, the measurement of acceleration has been considered. The method has been broadly used for the reason that a standard point such as measurement of displacement is not necessary. Acceleration data can be directly used to estimate stiffness of structures. Here, it is simply imagined that the acceleration data is also available to calculate displacement theoretically.

If we can combine the measuring method of acceleration to the concept of normalized deflection as well as the curvature, the proposed index is more reasonable in reality. For measuring of slab, there is a prospective problem about the effect of main frame vibration. It will be examined by simulating fatigue of slabs with main frame in future. Further study is still required.

5. CONCLUSION

1. Nonlinear finite element analysis which has been developed by Maekawa *et al.* can successfully reproduced the damage progress of RC slabs subjected to high cycle moving load.
2. Based on this study, it is suggested that the stiffness of 70mm thick asphalt pavement has a certain effect to the fatigue life of slab.
3. The normalized deflection "$d/\ d_0$" was proposed as a simple index to know the dangerous state of RC slabs. When the $d/\ d_0$ exceeds 3.0, more frequent monitoring is recommended.

4. The curvature “$1/r_x$” was proposed as a simple index to know the dangerous state of RC slabs. If the degree of $1/r_x$ starts to increase, it means that the damage progress is accelerated.
5. The measurement of acceleration is valuable to discuss acceleration data itself as well as to estimate displacement.

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