FATIGUE LIFE ASSESSMENT OF EXISTING BRIDGE DECKS BASED ON VISUAL INSPECTION DATA AND NUMERICAL SIMULATION

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***ABSTRACT*:** This paper presents a concept of the life prediction system for existing bridge decks based on a numerical simulation. First, crack information obtained in site investigation is converted to equivalent strains upon finite element model in order to reproduce a damage state of existing bridge deck. Then, the proposed system starts to simulate a residual life of existing bridge deck only based on the equivalent strains despite of the cause of cracks. The methodology was examined by sample data prepared for this study. As a result, it is suggested that the mid-span deflections at the beginning of simulation tend to be underestimated and they also tend to be overestimated when they get close to the failure. The numbers of load passage at failure might be reasonably predicted by this method from the practical view point.

KEYWORDS: bridge deck, life prediction, equivalent strain

1. INTRODUCTION

The maintenance of bridge is the one of current concerns of engineers. In particular, the problems of decks such as excessive deflection, steps on the joint and potholes on the surface, can be a cause of traffic accident, even if the main structure of bridge itself is healthy. Therefore, the efficient maintenance of bridge decks should be carefully discussed.

The efficient planning of maintenance needs a reasonable prediction of residual life of existing deck. Then, researchers have been developing the theoretical model for fatigue of concrete. Papa *et al*. and Alliche presented theoretical models for fatigue of plain concrete based on damage mechanics (1996, 2004). Matsumoto *et al*. successfully showed characteristics of cumulative damage of concrete by using Rigid Body Spring Model (RBSM) (2008) and it agreed with the past experiment done by Oh *et al*.(1991). For the structural concrete, Peerapong *et al*. showed a fatigue analysis of bridge deck using fracture mechanics (2006). Among those computer analyses, finite element analysis is probably one of the most realistic approaches. Palermo *et al*. had developed finite element analysis for cyclically loaded concrete structure (2007). The authors also have been developing the nonlinear finite element analysis that fully traces mechanical damage and plasticity of concrete under high cycle repetition of loads in use of logarithmic accelerated time integration (Maekawa *et al*. 2008). The computational framework developed by the authors has already been verified under high-cycle and dynamic loads (Maekawa *et al*. 2006a, 2006b).

It might be the right time to pay more attention to implement theoretical models into practical application as well. Here, the authors start to tackle with activating FE analysis for the life prediction of existing bridge decks. This project has two major policies. One is to utilize the data obtained in site investigation to the advanced numerical simulation system. Another is to serve the advanced numerical simulation system to the practical engineers with reasonable simplification. This paper presents a concept and methodology of the project.

2. METHODOLOGY

2.1 Flow of life prediction system

Since it is hard for engineers to record precise traffic history in the whole life of bridge, the proposed simulation system does not request them as input data. It predicts the residual life of existing bridge deck alternatively based on the data obtained from current inspection.

The flow of proposed simulation system is shown in Figure 1. The system needs current crack information as input. For example, crack spacing, width and length are needed in spite of the causes of cracks. It is also recommended to conduct additional inspections such as core sampling, radar exploration of rebar inside concrete even if the design document and drawings are kept well. The structural and material properties of existing bridge are possibly different from that had been designed.

2.2 Reproduce damage state based on inspection

Obtained crack information is automatically converted to equivalent average strain of concrete element with restraint of all nodes. In other words, the internal tensile stresses are given to each element. As a result, crack occurs in each targeted element. In this method, stress-strain relation traces the one obtained under monotonic and static load condition, even if the observed crack was generated by high-cycle repetition of load or high intensity of static load. In reality, various histories can leave a certain residual strain (see Figure 2).

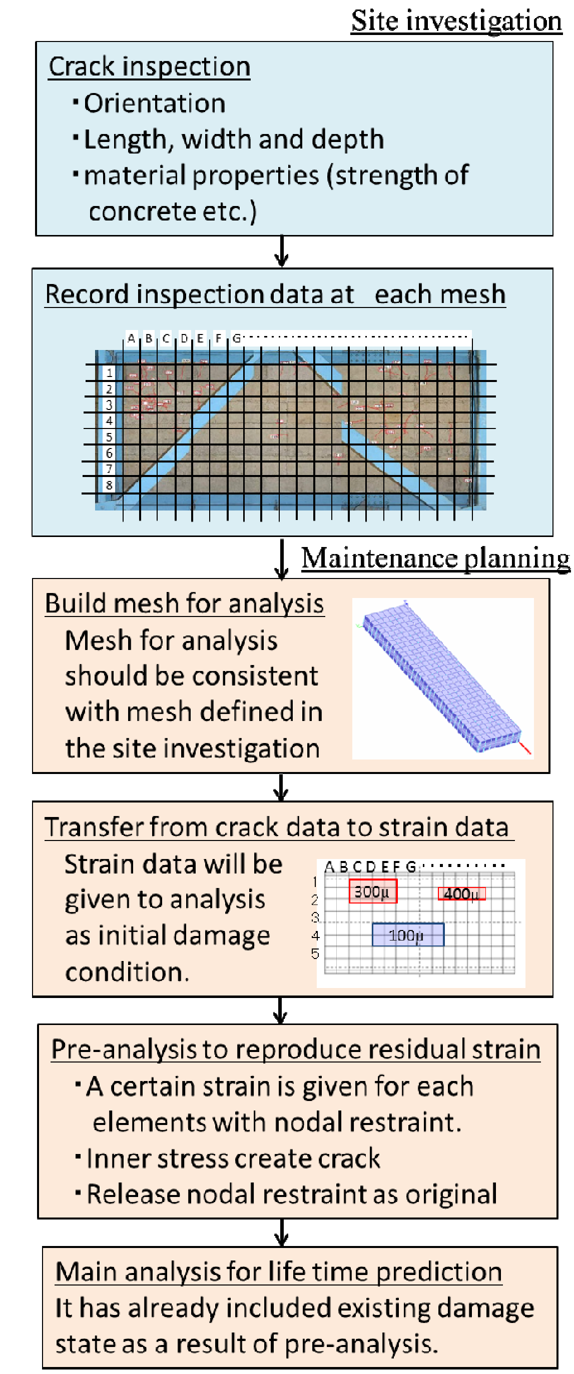


Figure1 Flow of life prediction system for reinforced concrete bridge deck

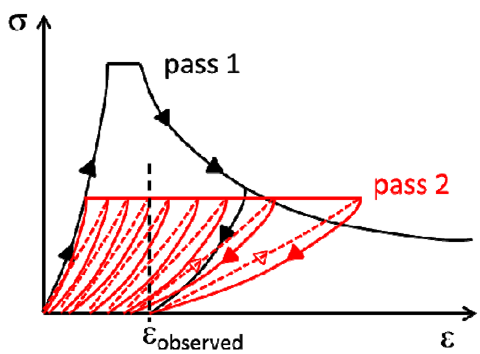


Figure2 Stress-strain relation for concrete in uniaxial tension

It means that this method can reproduce the degree of plasticity but does not always reflect the decrease of stiffness. However, the errors of stiffness at this moment are not dominant issue to predict future fatigue life. Because the constitutive laws used in this study estimates the fatigue damage progress based on the stress amplitude and the number of repetition. This will be mentioned at following section.

2.3 Fatigue damage progress in proposed method

It is widely known that fatigue of structural materials is generally presented as a function of stress amplitude and the number of repetition. The constitutive laws used in this study take into account the stress amplitude as a function of elastic strain. It is formulated in terms of uniaxial compression, tension and shear transfer across crack, respectively (Maekawa *et al*. 2008).

The stress-strain relation of compressive concrete is modeled by elasto-plastic and fracture concept (El Kashif and Maekawa.2004) as,

$$\varepsilon = \varepsilon_e + \varepsilon_p \quad (1)$$

$$\sigma = E_0 \varepsilon_e K_c \quad (2)$$

where, total strain represented ε is a summation of elastic strain ε_e and plastic strain ε_p. This is the concept of elasto-plastic model. Total stress σ is a simple multiplication of initial stiffness E_0, elastic strain ε_e and fracture parameter K_c. The damage evolution due to repetition of load is implemented in fracture parameter K_c as follows.

$$dK_c = \left(\frac{\partial K_c}{\partial t}\right)dt + \left(\frac{\partial K_c}{\partial \varepsilon_e}\right)d\varepsilon_e \quad (3)$$

$$d\varepsilon_p = \left(\frac{\partial \varepsilon_p}{\partial t}\right)dt + \left(\frac{\partial \varepsilon_p}{\partial \varepsilon_e}\right)d\varepsilon_e \quad (4)$$

In terms of the elastic strain ε_e, it reflects stress intensity (equation (2)). In other words, elastic strain can be an indicator of amplitude of load. The derivative shown above depends on time domain *t* as well. However, the time dependency of fracture parameter is not dominant under low stress amplitude that is thought to be fatigue state. Therefore, fracture process due to fatigue can be reasonably described by using instantaneous elastic strain regardless of past history, even if the K_c at the starting of calculation is a bit different from that of existing structure. The same story is here for the evolution of plastic strain.

Another stress-strain relation of tensile concrete incorporating with tension creep effect for RC members is proposed as,

$$\sigma = E_0 \varepsilon K_T \quad (5)$$

where, ε is total strain including shrinkage and K_T is a fracture parameter for tensile concrete. The evolution of fracture parameter K_T is presented in similar manner of compressive concrete as,

$$dK_T = Fdt + Gd\varepsilon + Hd\varepsilon \quad (6)$$

where, derivative *F* indicates time dependent fracture due to tension creep effect. *H* stands for instantaneous fracture related to the tension softening. Fracturing due to repetition of load which is the concern of this study is implemented into *H* that is a function of strain.

Shear transferred across crack is formulated by crack width and slip. The formulation was derived from the mechanics of crack roughness and contact friction. Therefore, it is supposed that fatigue of shear transfer across crack is proportional to the number of cyclic load due to smoothing of rough crack surface and we have

$$\tau = X \cdot \tau_{or}(\delta, \omega) \quad (7)$$

$$X = 1 - \frac{1}{10}\log_{10}\left(1 + \int \left|d(\delta/\omega)\right|\right) \quad (8)$$

where, τ is transferred shear stress under the high cycle load, τ_{or} is the transferred shear stress calculated by original contact density model (Li and Maekawa. 1987), ω and δ indicate crack width and slip, respectively. X is the fatigue modification factor related with accumulation of shear deformation during the loading. X changes in regard to common logarithmic scale. For example, when the true value of common logarithm is null, factor X is 1.0, and when the true value is one billion, factor X is 0.1. This indicates that reduction of shear stiffness expressed by factor X is highly dependent on the number of load repetition, because the slip normalized width cannot increase in logarithmic at each cycle.

3. Verification

3.1 Bridge deck model for FE analysis

Since there seems not to have any available data set which consists of crack information obtained by site investigation and data of fatigue loading test for the investigated slab, the authors try to verify the proposed method only by using analysis.

The reinforced concrete slab model is shown in Figure 3. The plane dimensions of the slab are 2.1 m x 4.5 m and 190 mm thick. The slab is directly supported by two longitudinal lines and the main span is 1.8 m. Single layers of reinforcement are provided on the both upper and lower layer with the assumption of 30mm cover depth for each. Rebar arrangement of both sides is shown in Table 1. Because of its symmetry, only half of the slab is modeled in the analysis. Solid elements are used throughout for the model in which material property is presented in Table 2. The total numbers of nodes are 1,296, and 920 solid elements are arranged for the 3D analysis domain.

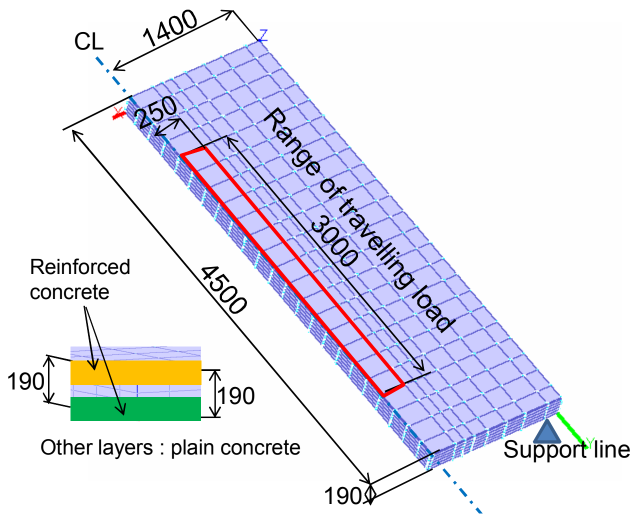


Figure3 Reinforced concrete slab model for verification

Table1 Rebar arrangement of studied model

Element	Transverse	Longitudinal
Upper layer	D16ctc300	D13ctc500
Lower layer	D16ctc150	D13ctc300

Table 2 Material properties of studied model

(N/mm^2)	Compressive strength	Tensile strength	Young's modulus
Concrete	24.0	1.15	25,000
Rebar	---	345	210,000

Table 3 loading variation

Case		Loading pattern	Purpose
1		Moving load until fail	Prototype
2	-L	Moving load 2,000 passages	Obtain Light damage state
	-H	Moving load 200,000 passages	Obtain Heavy damage state
3	-L	Moving load until fail With light damage	Comparison 1
	-H	Moving load until fail With heavy damage	Comparison 2

The moving point load rolls back and forth in a range of 3.0 m as shown Figure 3. 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 80 kN during the passage of moving loads but the gravity center of the applied nodal forces moves step-by-step.

3.2 Extract damage state at a certain point

The progress of mid-span deflection is traced in Figure 4. Generally speaking, damage progress of reinforced concrete structure could be explained by three damage stages. At the first stage, displacement or deformation of structure increase rapidly due to occurring micro cracks. At the second stage, the increment of deflection becomes small and constant with the number of passage in terms of logarithmic scale. Created cracks are gradually developed at this stage. Then, the displacement or deformation of structure again starts to increase rapidly due to the progressive damage just before the failure. This is thought to be a final stage.

However, it is not easy to clarify the three damage stage in Figure 4, the authors try to extract two different damage stage. The case 2-L and 2-H are intended to represent the second stage and the third stage, respectively. The residual strains are saved after 2,000 passages as the case 2-L and 200,000 passages as the case 2-H (see Table 3).

Figure5 shows the contour maps of residual strain of case 2-L. In terms of transverse direction, relatively high residual strains are observed under the area of moving load (Figure 5(a)). In terms of longitudinal direction, relatively high residual strains are observed at the edge of the moving load (Figure 5(b)). Figure 6 shows the case 2-H. the trend is similar to the case 2-L, however the values are larger than that of the case 2-L.

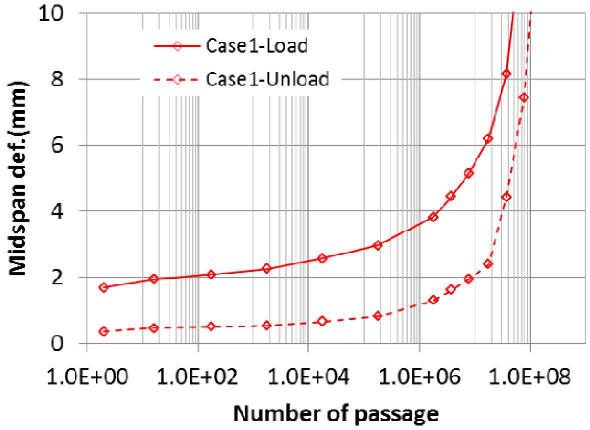
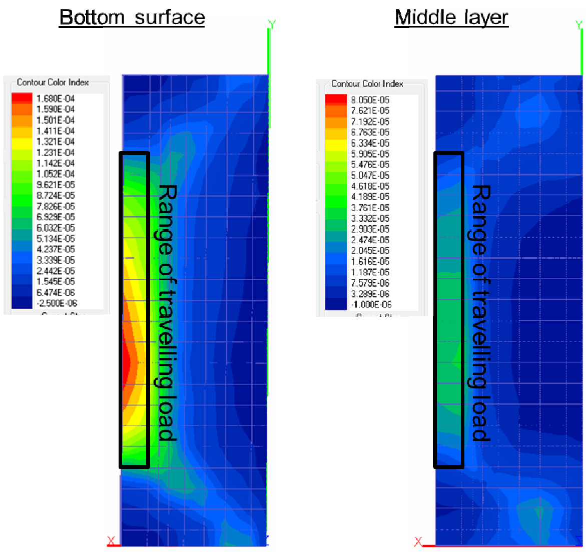
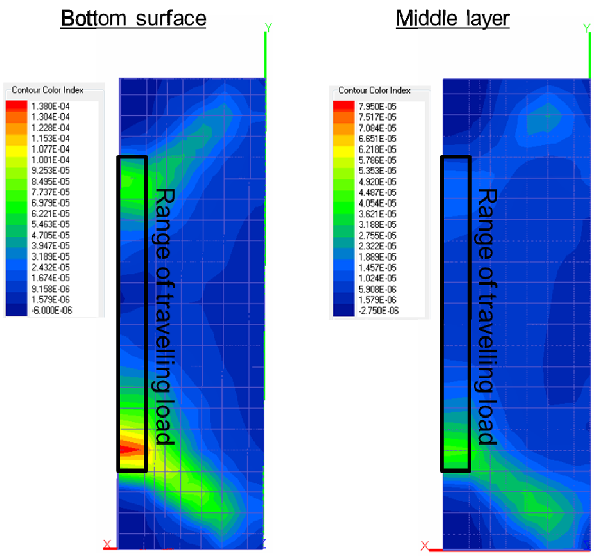


Figure4 Progress of mid-span deflection (case1)

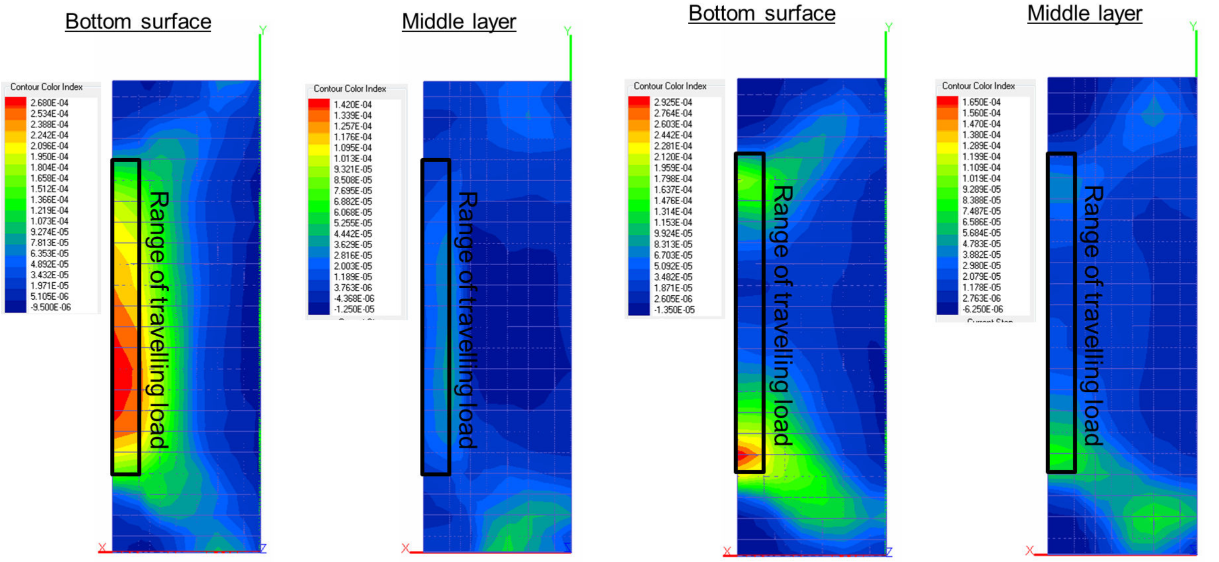


(a) Residual strain in X (transverse) direction



(b) Residual strain in Y (longidudinal) direction

Figure5 Contour of residual strains (case2-L)

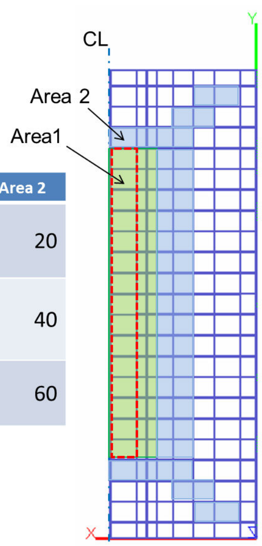


(a) Residual strain in X (transverse) direction (b) Residual strain in Y (longitudinal) direction

Figure6 Contour of residual strains (case2-H)

Transverse(X)

Strain (μ)	Area 1	Area 2
Middle	50	20
Second from bottom	110	40
Bottom	170	60



Longitudinal(Y)

Strain (μ)	Area 1
Middle	50
Second from bottom	75
Bottom	100

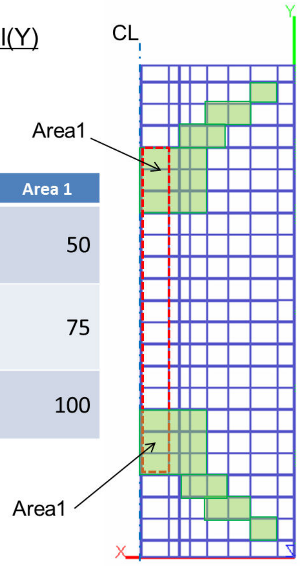
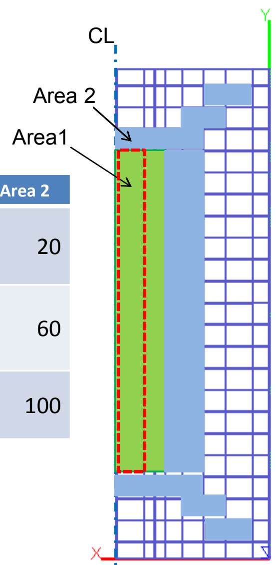


Figure7 Input strains for pre-analysis (Case3-L)

Transverse(X)

Strain (⊦)	Area 1	Area 2
Middle	70	20
Second from bottom	155	60
Bottom	240	100



Longitudinal(Y)

Strain ( )	Area 1
Middle	70
Second from bottom	120
Bottom	170

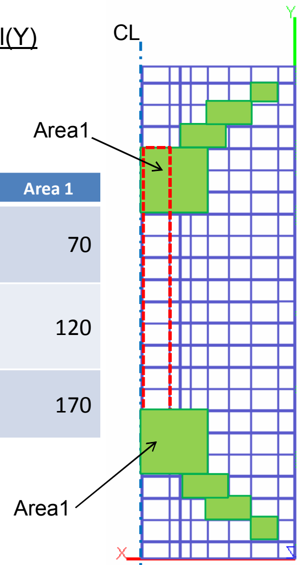


Figure8 Input strains for pre-analysis (Case3-H)

3.3 Comparison of predicted residual lives

In this study, the computation gives the profile of residual strain as Figure 5 and 6. However, it is not easy to obtain precise crack information from site investigations.

According to technical report on NETIS, the possible accuracy for measuring crack width can be thought 0.02 mm in site. Therefore, the authors adopt 100 μ as minimum value of residual strain at the bottom surface of slab model with considering the mesh size of the slab model in this study. 100 μ is equivalent to 0.02 mm width single crack in a 200 mm plane square element.

The strains used for pre-analysis are show in Figure 7 and Figure 8. Residual strains calculated less than 100 μ are omitted. With these strain profiles, life prediction simulations are conducted as the case 3-L and 3-H, respectively.

The progress of mid-span deflections is plotted together in Figure 9. In terms of the case3-L, the progress of mid-span deflection both at loading and at unloading show agreements with those obtained for case1. On the contrary, for the case3-H, the mid-span deflection at loading tends to be underestimated at the beginning and that also tends to be overestimated at the end of its life. Those differences possibly come from the errors of estimated stiffness at the beginning of simulation. Progressive damage of concrete might have already started at the starting point of simulation case3-H. At this stage, it is not easy to synchronize progressive damages in the proposed method.

The other hand, each number of passage at failure does not have substantial differences from the practical view point. It is suggested that the constitutive laws reasonably reproduce sequential damage progress even if the estimated stiffness at the beginning have some errors. This indicates that the proposed method works well when the damage progress in long term is more dominant than the errors of stiffness at the beginning.

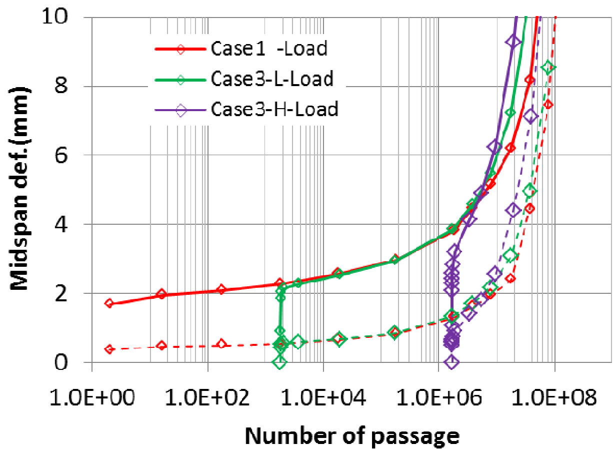


Figure 9 Progress of mid-span deflections

Through this study, it can be said that the proposed method might be able to offer a reasonable prediction of fatigue life for existing structure. In order to authenticate the limitation of proposed method, further simulations by using data obtained at site investigations is required in future.

4. CONCLUSIONS

This paper presents a concept of the life prediction system for existing bridges' decks based on a numerical simulation.

1. In order to reproduce a damage state of existing bridge deck, the equivalent strain is given to the simulation as initial input. The equivalent strain is calculated from crack inspection data obtained on site.
2. At the first loading, the mid-span deflection obtained by proposed method does not always show good agreement with original one. The differences possibly come from the errors of estimated stiffness.
3. On the other hand, the number of passage at

failure obtained by proposed method shows reasonable agreement with original one. This might mean that the constitutive laws successfully reproduce lasting damage progress even if the estimated stiffness at the beginning has some errors.

4. On the basis of this study, the proposed method might be able to offer a reasonable prediction of fatigue life for existing structure for the case of low intensity and high cycle fatigue problem.

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