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REMAINING LIFE AND CORROSION LIMIT OF CORRODED TRUSS BRIDGE FOCUSING ON THE ULTIMATE STRENGTH

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This study is targeted for the estimation method for the remaining life of a corroded truss bridge based on the ultimate strength through one of case studies. The remaining life can be estimated by the corrosion limit which is defined as a corrosion state based on the assumption that the bridge collapse occurs when reaching the load required of the bridge performance. The deterioration curve of the corroded truss bridge is proposed by conducting nonlinear finite element analyses, in which the overall truss bridge is modeled as a rigid frame which consists of beam elements. According to the analytical results in this study, the overall bridge model at a non-corroded state has the safety factor as about 5 times of the design live load. In general, almost actual bridges have the concealed safety factors which were not considered in the design. It can be said that this is the reason why the bridges don't collapse even if the severe corrosion occurs in members. However, corrosion progressions cause the strength reductions of bridges, then the bridges finally collapse when they reach the corrosion limit. Therefore, this study shows the fact that it is very important to evaluate the current safety of the actual bridges quantitatively by estimating the corrosion limit and remaining life.

Keywords: Ultimate strength, Remaining strength, Nonlinear FEM analysis, Deterioration curve, Overall structure, Maintenance.

1 INTRODUCTION

Recently in Japan, there are a lot of steel bridges with conspicuous corrosion due to the aging of themselves. 20 years later from now, over 70% of approximately 700,000 existing road bridges with at least 2 m in length will reach the age of more than 50 years. As a result, deteriorated bridges are thought to increase even more, at the same time, in light of recent socioeconomic conditions, they would be needed to maintain more rationally and efficiently to ensure the sustainable use and the safety. Thus, it is required for civil engineers to evaluate the safety of corroded bridges based on the load bearing capacity. However, in fact, the safety of bridges has usually been evaluated by visual inspections without judgment based on a quantitative way, such as the remaining strength. It is very difficult for engineers to evaluate the corroded bridges numerically according to the corrosion progression because they often encounter the following questions; 1) How to evaluate the remaining load bearing capacity of corroded bridges, 2) How to judge the extent of corrosion damages can be allowed, and 3) How to estimate the remaining life of existing bridges. As an available method for resolving the questions above in terms of the maintenance field, it is considered that the safety of structures can be evaluated based on the strength of overall bridges. This is why the bridges actually exist as total systems that consist of various structural members, although the bridge design usually is based on the design for each member.

In this study, the remaining strength and corrosion limit of corroded truss bridges are analyzed and evaluated based on the ultimate strength and behavior of the 3D model, and the remaining life is estimated by deciding the required performance. Finite element analyses are performed considering geometrical and material nonlinearities, then the rationality and applicability of the overall 3D model is shown by comparing the results of both models.

2 METHOD FOR ESTIMATING THE REMAINING LOAD BEARING CAPACITY, CORROSION LIMIT, AND REMAINING LIFE

First, in order to estimate the load bearing capacity of a truss bridge at the present situation, an analytical model which can express current corrosion conditions is made up. In this study, the remaining load bearing capacity as the ultimate strength can be obtained by FEM analyses under the conditions of incremental loads by adding the live loads to the dead load. Corrosion damages are given to the model by thickness reductions of steel plates which compose members. Then, it is assumed that the corrosion damages increase proportionally according to the elapsed time. From the analytical results accompanied with the corrosion damages, the relationships between thickness reductions and remaining load bearing capacities can be obtained. The corrosion limit would be given as the minimum values of remaining steel plate thicknesses when meeting the required performance, then the required strength of the truss bridge is assumed that it can bear the load as three times of the live load used for the original design. Finally, it is estimated the elapsed years to reach the corrosion limit by using the relationships between the exposure time and the thickness reductions of steel plates. The remaining life is defined as the difference between the elapsed years to reach the corrosion limit and the elapsed years at the time.

The corrosion state is shown by the thickness reduction ratio R as shown in Eq. (1).

$$R = \frac{\text{Reduction of cross sectional area}}{\text{Original cross sectional area}} \times 100\ [\%] \tag{1}$$

In other words, '$R=R_n$%' indicates the status that the thicknesses of every steel plates composed each member section are reduced by Rn% respectively. 'R=100%' means a non-existence of elements, that is to say, a member fracture. In addition, it is used a parameter called the reduction magnification ratio β as one corrosion index of the truss bridge because the thickness reduction ratios are different for each member. 'β=1' indicates the current corrosion state, the thickness reduction ratios R of each member increase with proportional factors of the reduction magnification ratios β.

3 CONDITIONS OF ANALYSIS

Skeleton structures and respective member names of the bridge applied in this study are shown in Figure 1. This shows a steel Warren truss bridge that has been designed according to the Japanese specifications for highway bridges.

In order to obtain the corrosion limit, the ultimate strength of the overall bridge is calculated by the large deformation elastic-plastic finite element analysis performed with ABAQUS, a general-purpose FEM analysis program by overall (3D) bridge model Figure 2. The bridge model is made by reference to the literature (Tetsuya Nonaka et al, 2010), where the steel and deck parts are modeled using beam elements and laminated shell elements consisting of concrete layers and rebar layers respectively. The material constitutive law of the steel is shown in Figure 3 based on Eq. (2) as the formula of Ludwik.

$$\sigma = \begin{cases} 20000\,\varepsilon & (\varepsilon \geq \varepsilon_y) \\ 235 + 500\,\varepsilon^{0.5} & (\varepsilon < \varepsilon_y) \end{cases} \tag{2}$$

Herein, ε_y indicates a yield strain of the steel. Figure 4 shows the constitutive laws related to the RC slab that take into account the rebar tension stiffening effect and the adhesion of the rebar and concrete (Japan Society of Civil Engineers, 2013). Slab anchors of connections between the concrete slab and the steel girder are expressed as non-linear springs. The relationship between horizontal slip forces and relative slip displacements is shown in Figure 5. For boundary conditions, one end has a pinned bearing, the other end has a roller bearing.

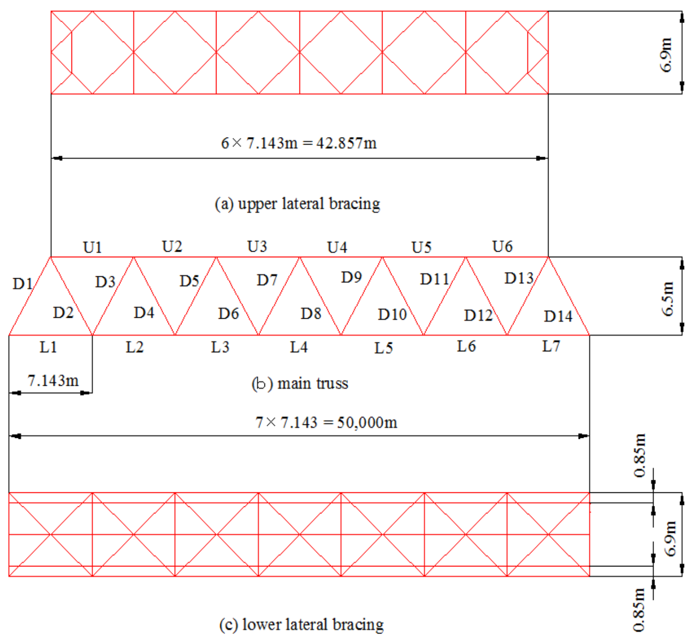


Figure 1. Skeleton and respective member names of bridge.

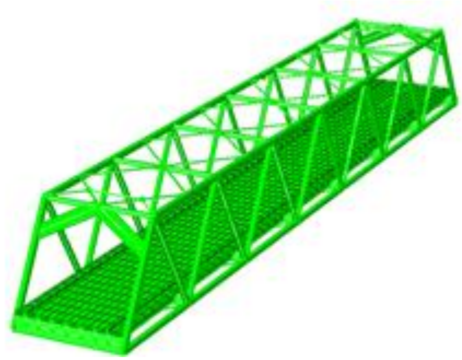
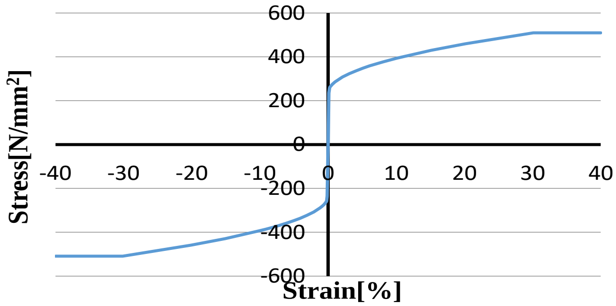
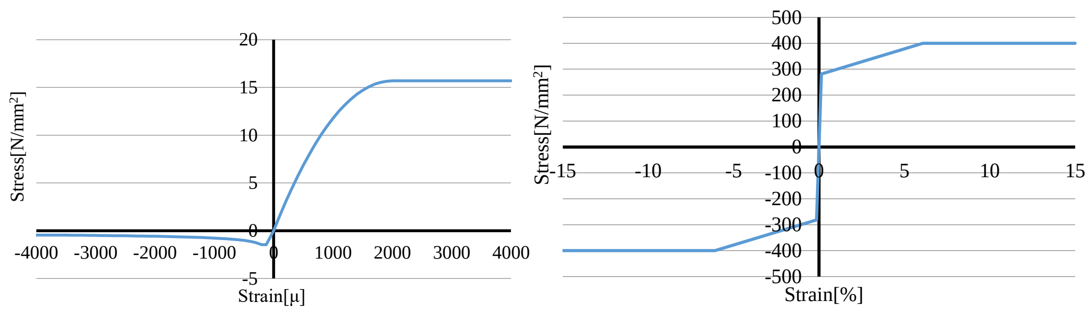


Figure 2. Overall (3D) bridge model.



Modulus of elasticity [N/mm^2]	200000
Yield stress [N/mm2]	235
Poisson's ratio	0.3
Unit weight [kN/m^3]	77

Figure 3. Material constitutive law of steel.



(a) Concrete layer (b) Rebar layer

Figure 4. Material constitutive law of RC slab.

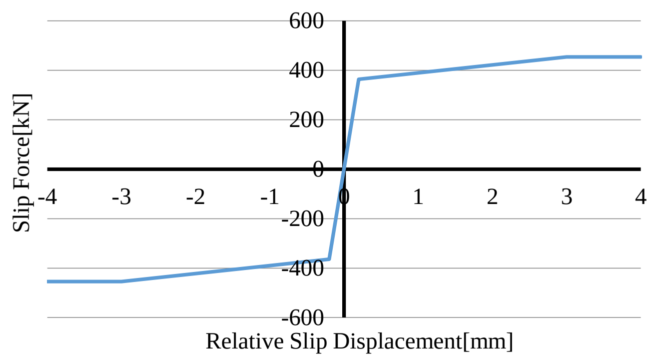


Figure 5. Relationship of horizontal slip force and relative slip displacement of slab anchor.

First, the dead load L_d is loaded, then the design live loads L_l increase gradually to the extent of the maximum load by using load increment parameters α as shown in Eq. (3), where α is called the live load magnification and shows a safety factor against the design live load. In this study, the arrangement of the live load is set as the situation that the member force of L3 is maximized.

$$L_{all} = L_d + \alpha L_l \tag{3}$$

To estimate the ultimate strength, corrosion limit and remaining life, the following assumptive situations are applied to this bridge model; for the thickness reduction ratios R, the

D4, L2, L3 members are given as 10%, and the D5 member is given as 20%, under the condition that 15 years have passed since the beginning of bridge use.

4 ESTIMATIONS OF THE REMAINING LOAD BEARING CAPACITY, CORROSION LIMIT, AND REMAINING LIFE

In order to evaluate the corrosion limit, the live load magnifications α at the ultimate strengths of the 3D bridge model are estimated by analyzing the total 10 cases that the thickness reduction rates R are gradually increased by the reduction magnifications β as shown in Table 1. The relationships of the reduction magnifications and the ultimate strengths are shown in Figure 6. According to the figure, when α is equal to 3 that was defined as the corrosion limit of the bridge, β is almost equal to 6.7. That is to say, it can be said that the bridge may lose a structural functional capacity when β reaches almost 6.7 (R of the D4, L2, L3 members is 67%, R of the D5 member is 100%). Then the relationships between the exposure time and the thickness reductions of steel plates with consideration for the anti-corrosion effect of coating film and its degradation can be given as Figure 7 based on the corrosion surface simulation model (Katashi Fujii et al, 2004). According to the figure, the elapsed time to reach the corrosion limit as the reduction magnification "β=6.7" is around 31 years. Thus, the remaining service life is almost 16 years because nearly 15 years have already passed.

Furthermore, Figure 8 shows the bridge can secure the safety factor as about four times of the design live load even though the D5 member completely fractures with the reduction magnification "β=5". However, it is to be noted as fact that the strength itself has been decreasing as compared to the sound condition.

Table 1. Corrosion state of each analysis model.

Corroded Member	Reduction Magnification β									
	1	2	3	4	5	6	7	8	9	10
	Thickness Reduction Rate R [%]									
L2, L3, D4	10	20	30	40	50	60	70	80	90	100
D5	20	40	60	80	100	100	100	100	100	100

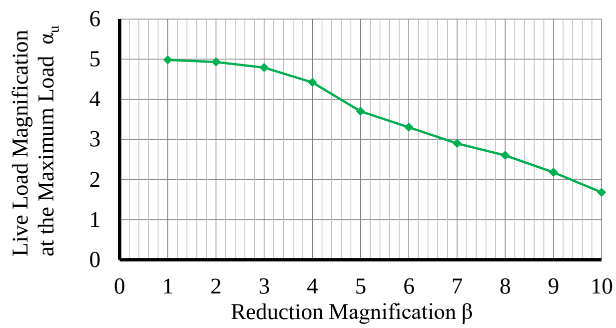


Figure 6. Relationships between reduction magnifications β and ultimate strengths.

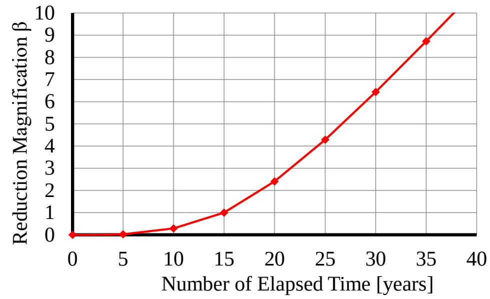


Figure 7. Relationship of elapse time and reduction magnifications *β*.

5 CONCLUSIONS

(1) By reference to the literature 1), the steel Warren truss bridge is modeled that beam elements and laminated shell elements are applied to the steel and deck parts respectively. The ultimate strengths of the overall bridge under several corroded conditions are calculated by using the composite nonlinear analytical method with consideration for the material and geometric nonlinearity.

(2) The corrosion limit of the corroded truss bridge would be given as the minimum values of remaining steel plate thicknesses when meeting the required performance, then the required strength of the truss bridge is defined that it can bear the load as three times of the live load used for the original design. It is estimated the elapsed years to reach the corrosion limit by using the relationships between the exposure time and the thickness reductions of steel plates. The remaining life is defined as the difference between the elapsed years to reach the corrosion limit and the elapsed years at the time.

(3) According to the analytical results in this study, the overall bridge model at a non-corroded state has the safety factor as about 5 times of the design live load. Then the bridge can secure the safety factor as about four times of the design live load even though the D5 member completely fractures with the reduction magnification "β=5". This fact indicates that actual bridges have the concealed safety factors which were not considered in the design. It can be said that this is the reason why the bridges don't collapse even if the severe corrosion occurs in members. Therefore, it is very important for a near future maintenance theory to evaluate the current safety of the actual bridges quantitatively by estimating the corrosion limit and remaining life.

References

Tetsuya Nonaka, Tsutomu Usami, Maki Iwamura, Atsushi Hirozumi, and Hiroichi Yoshino, A *Proposal for a Redundancy Analysis of Steel Bridges including Progressive Member Failures*, Japanese Journal of Structural Engineering, Vol.56A, 779-791, March, 2010.

Japan Society of Civil Engineers, *STANDARD SPECIFICATIONS FOR CONCRETE STRUCTURES-2012*, Design, Maruzen Co., Ltd., Japan, March, 2013.

Katashi Fuji, Tatsumasa Kaita, Hideharu Nakamura, and Ichiro Ario, *A model generating surface irregularities with consideration of corrosion progress in aging*, Japanese Journal of Structural Engineering, Vol.50A, 657-665, March, 2004.