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REMAINING STRENGTH EVALUATION OF FERRY BRIDGE CORRODED LOCALLY

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This paper presents the results of a nonlinear finite element analysis to determine the remaining ultimate strength of a ferry bridge with local corrosion at the girder flange. The ultimate strength decreases with the corrosion thinning of the flange, when the corrosion damage locates at the center of the span. On the other hand, when the corrosion damage exists near the support, the ultimate strength did not decrease, which shows the repair of the flange thinning near the support is not so effective in strength rehabilitation of the bridge. The reason of these facts is; the collapse of the bridge is due to bending moment in the former case, but in the latter, it is due to not bending moment but shear force, therefore, flange thinning does not concern with the shear strength. Thus, rational maintenance will be possible after clarifying the collapse mode and the remaining strength of the bridge.

Keywords: Plate girder, Ultimate remaining strength, Nonlinear FEM analysis, Safety factor of ultimate load, Flange local corrosion.

1 INTRODUCTION

In Japan, there has been a growing consciousness of importance of maintenance because serious damages have been appearing in a lot of deteriorated infrastructures constructed from 1960s to 1980s. The proportion of bridges exceeding 50 years since construction is currently about 20%, but it will increase to about 40% in 10 years and to about 70% in 20 years, consequently, to prolong the life of bridge is becoming an urgent and grave task, when the future serious economic condition is taken into account.

Although soundness of deteriorated bridge has been usually being judged currently from the corrosion-damaged grade, the safety measures decided by that judgement may not necessarily be appropriate in terms of strength and cost, from the point of whole bridge. Therefore, it will be more appropriate in the future to maintain based on the remaining strength of the bridge.

According to the above concept, the remaining strength of a whole bridge has been investigated by Tai *et al.* (2015), Yamaguchi *et al.* (2011) and Connor *et al.* (2005) who evaluated the ultimate remaining strength of general road bridges corroded by FEM. However, in their researches, the effect of the corrosion location and grade on ultimate remaining strength has not been clarified yet. By conducting nonlinear finite element analysis considered with geometrical and material nonlinearities, the ultimate remaining strength of ferry bridge as well as that of general road bridge built with steel girders has been investigated by Takami and Fujii (2014, 2017). Then, it was revealed that the ultimate remaining strength of the ferry bridge is far lower than that of general road bridge, because the ferry bridge is a simple structure consisted of main girder and steel floor deck and has few secondary members. Adding to this, ferry bridge is in the environment that is easier to corrode than general road bridge, because it is constructed at sea shore in most cases.

Therefore, it seems more important to grasp ultimate behavior and remaining strength of ferry bridge than road bridge. In this paper, conducting nonlinear finite element analyses, ultimate behavior and remaining strength are investigated when the local corrosion progresses at the tensile flange of the main girder, varying the location and the grade of the corrosion.

2 ANALYSIS MODEL OVERVIEW

The bridge to be analyzed is a movable ferry bridge completed in 1977 at Fukuyama Port which locates in the southeastern area of Hiroshima prefecture. The dimensions of the bridge are shown in Figure 1. The bridge length is 20.6 m, the span is 20.0 m, the effective width of the steel deck is 4.0 m, the girder height is 1.3 m, and the checker plate with a thickness of 12 mm is used for the floor slab. The main girder web and the cross beam are joined by welding and the steel floor deck is attached by welding on the cross beam. Vertical ribs are attached to the steel floor slab by welding at approximately 340 mm intervals. As shown in Figure 1, in section A and C, each cross-section changes because the flange has a gradient.

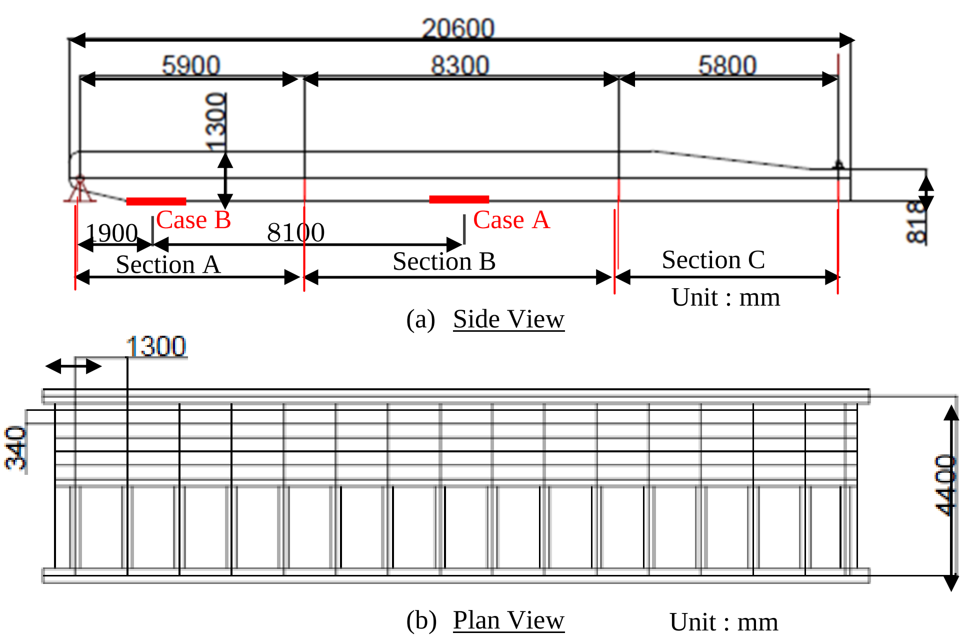


Figure 1. Dimensions of analyzed ferry bridge.

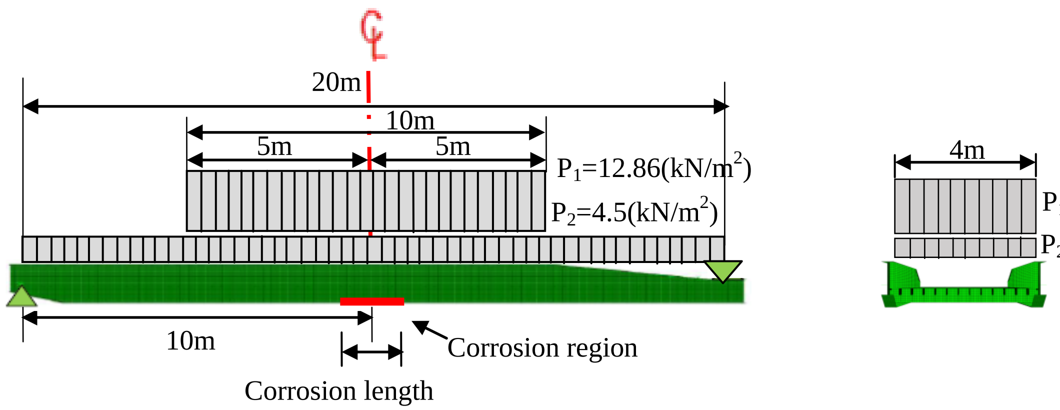
Nonlinear FEM analysis is conducted on this bridge using ABAQUS. All steel members are modeled by using 4-node iso-parametoric shell element, and the supports at both ends are modeled by using rigid element. The material properties of steel member are assumed as perfectly elasto-plastic material and are shown in Table 1. The support on the ground side is a pin support allows only the rotation around the axis of the direction perpendicular to the bridge axis, and the support on the sea side is a suspension support restrains only the vertical displacement.

Table 1. Materials properties of steel member.

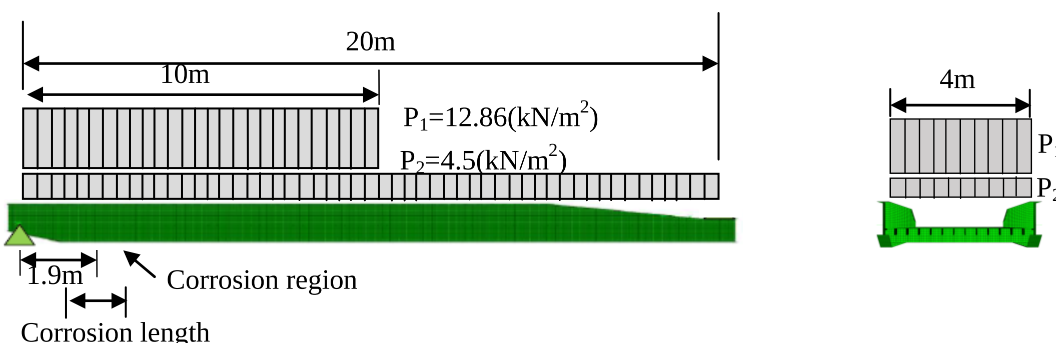
Young's Modules [N/mm^2]	Yield Stress [N/mm^2]	Poisson's Ratio [-]	Unit Weight [kN/m^3]
210000	253.2	0.3	78.5

The dead load L_d is subjected to firstly, and then the live load L_l is subjected and increased up to the maximum load, consequently, the total load L_{all} is given by Eq. (1) in which α in Eq. (1) is the load increment parameter. Since α can be regarded as a kind of safety factor on live load, we call α safety factor hereafter. According to Japanese Specifications for Highway Bridges (JSHB), two kind of distributed loads (live load) P_1 and P_2 are subjected on the floor deck in order that the location of corrosion damage becomes most dangerous, as shown in Figure 2. Then, the location of live load changes depending on the location of local corrosion.

$$L_{all} = L_d + a \cdot L_l \tag{1}$$



(a) Case A (corrosion at the center of the span)



(b) Case B (corrosion near the girder end)

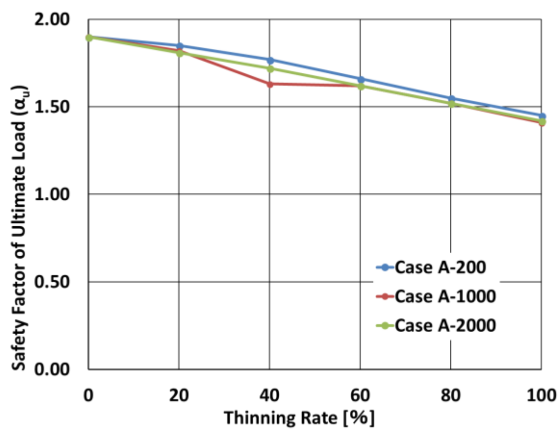
Figure 2. Location of live load (distributed L-load in JSBH).

In this paper, we assume that the location of local corrosion is at the center of the span and near the support shown by the red line in Figure 1 and 2, and let them be Case A and Case B, respectively. For the corrosion damage, the degree of corrosion is expressed by the thinning rate R shown in Eq. (2), and the thinning rate is changed from 0% to 100%. Moreover, the length of the corrosion is changed to 200 mm, 1000 mm, 2000 mm, respectively. Also, we assume the thickness of the flange is uniformly reduced from both surfaces, then the eccentricity of neutral plane does not change after corrosion.

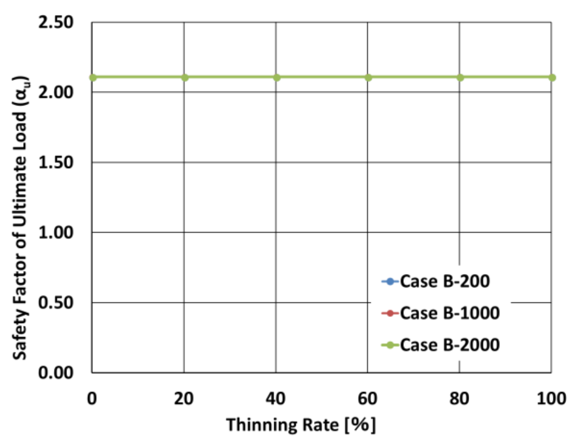
$$R = (\text{Reduced thickness of steel}) / (\text{Original thickness of steel}) \times 100[\%] \qquad (2)$$

3 ANALYSIS RESULT AND DISCUSSION

Figure 3 shows the relationship between the safety factor α_u at the maximum load and the thinning rate R. In the figure, "200" of "Case A-200", for example, means the length of corroded region of the local corrosion. It can be noticed from Figure 3 (a) that the safety factor α_u is 1.9 at the initial state without corrosion, which is significantly lower than general road bridges (usually α_u=4~5), and also that the safety factor decreases gradually with thinning progression. The figure shows the safety factor reduces to 1.45 when the lower flange at the center disappears. On the other hand, when the local corrosion locates near the support, it can be noticed that the safety factor does not change α_u=2.11 irrespective the thinning rate and also the length of corrosion region, as shown in Figure 3 (b).



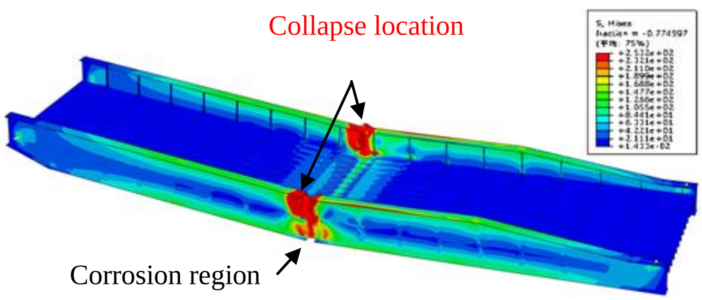
(a) Case A (corrosion at the center of the span)



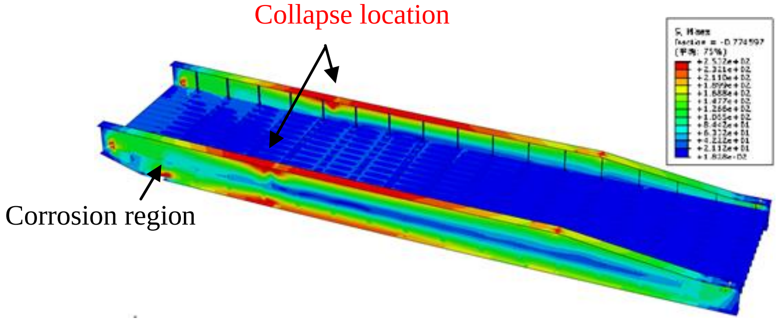
(b) Case B (corrosion near the girder end)

Figure 3. The relationship between safety factor and thinning rate.

The collapse state and the contour of von Mises stresses at the maximum load are shown in Figure 4. The collapse is caused by flange yielding in both cases. However, it is noticed that the location of flange yielding is different in both cases. Namely, at the center of the bridge in Case A, which is the same as the location of the local corrosion, but for Case B, the yielding occurs at the butt joint, where cross-sectional dimensions change, different from the corroded location. This is the reason that the safety factor does not concern with the corrosion thinning. Also, Case B shows the remaining strength does not enhance even though the corrosion is repaired. Thus, clarifying remaining strength seems to be important for the economical and efficient maintenance.



(a) Case A (corrosion at the center of the span)



(b) Case B (corrosion near the girder end)

Figure 4. Collapse mode and von Mises stresses at maximum load.

4 CONCLUSIONS

In this study, remaining strength of entire bridge was analyzed for a ferry bridge corroded locally, by nonlinear FEM. Local corrosion damage was given in the lower flange only of main girder, changing the location and the grade of corrosion. The following terms can be listed from analytical results.

(1) Although the corrosion damage makes the bridge strength decrease in almost case, the possibilities to be unrelated to ultimate remaining strength is shown as a case. If the location of local corrosion is different from the trigger point of the collapse, the ultimate strength may not decrease due to corrosion. Consequently, it is important for adequate bridge maintenance that ultimate behavior and collapse mode after corrosion are analyzed and clarified from the view point of whole bridge.

(2) The safety factor for live load was 1.9 at initial state, though the ferry bridge analyzed in this paper is a typical type. This value is significantly smaller than general road bridge which has at least 4 commonly. Adding to this, considering flying salt environment of ferry bridge, more carefulness will be necessary in maintenance of ferry bridge.

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