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Shear failure behaviors of a web panel with local corrosion depending on web boundary conditions

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ABSTRACT

In this study, the shear buckling behavior and shear failure mode of a locally corroded web were examined using nonlinear FE analysis, depending on the corroded web and web boundary conditions. Longitudinally corroded web panels and triangularly corroded webs were considered for quantitatively evaluating the shear buckling behaviors of the corroded web panel. For the longitudinally corroded web panels, the shear buckling strengths were only slightly reduced by about 9–16% depending on the corroded web conditions and web boundary conditions. Their diagonal tension field widened and became distorted with the pronounced shear–bent shape of the corroded web. In particular, in a web-flange separated boundary case, it is out-of-plane displacement and deformation exhibited different shear buckling modes, i.e., a diagonal tension field in its upper part and a large triangular lateral displacement in its lower part. On the other hand, the triangularly corroded web panels had only slight effect on the shear buckling resistance of the corroded web because this corroded model does not affect the diagonal tension field of web panel.

1. Introduction

In Japan, at least half the bridges with a span length exceeding 15 m can be evaluated to have a service period of over 50 years [1]. Thus, there is a growing interest in the maintenance of bridges with service periods of more than 30–50 years, for improving their durability and for establishing a method for evaluating their residual strength [1]. In the case of steel bridges, the fatigue cracks and corroded structural members can be important issues during maintenance, depending on their structural behaviors, details, and environmental conditions. The fatigue cracks of steel bridges are recognized as a major issue, and thus, various studies have been conducted for a long time. With regard to the corrosion of the steel bridges, however, it can be considered to be sufficient as its paint coating, which is generally used to prevent corrosion. However, as severely corroded steel bridges and failures have recently been reported, various studies on the corrosion phenomena and residual strengths of corroded steel bridges are being conducted as well as their reinforcement methods [2–8].

When corrosion occurs in a steel plate girder bridge, it can affect the bridge's load-carrying capacity, shear buckling strength, and bearing capacity. The flange thickness loss due to corrosion will reduce the net area available to resist bending, increase the displacement, and reduce the sectional properties. Therefore, the bridge's load-carrying capacity will be reduced, and the web thickness loss near the support will also reduce the net area available to resist shear buckling and axial loads. As such, the shear strength and bearing capacity will also be reduced, as shown in Fig. 1(a). Thus, studies on the evaluation of the residual strength of steel bridges, such as the residual shear strength of their web panels, their bearing capacity, and their load-carrying capacity, are being conducted [2–8].

Especially, for residual shear buckling strength of a web panel with local corrosion, Kim et al. conducted the shear loading test for a plate girder with local corrosion to examine the shear buckling behaviors of locally corroded web panel and estimate residual shear buckling strength depending on corrosion condition of web panels [7]. Ahn et al. compared the shear buckling behaviors and shear buckling failure modes of the plate girders with local corrosion and numerically examined residual shear buckling strengths of the plate girders with local corrosion under various corrosions conditions to propose the residual shear buckling strength reduction factor of the web panel with local corrosion [8]. However, the shear behavior and shear failure mode of corroded web panels have not yet been clearly examined, considering various corroded web conditions as shown in Fig. 1(b). Therefore, in this study, the shear behavior and shear failure mode of corroded web panels were evaluated depending on the corroded web patterns and web boundary conditions. To quantitatively clarify the

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shear behavior of a corroded web panel, nonlinear FE analyses were conducted. The out-of-plane displacement and the diagonal tension files in the web were compared to explain the corrosion effect on the shear behavior of the web.

2. Shear strength and behaviors of web panel and FE analysis model

2.1. Shear strength and buckling behavior of web panel

Various experimental and numerical approaches have been carried out to examine the shear buckling strength of web panels and their post buckling behaviors. Design equations of a web panel have also been suggested after research on the critical buckling strength of a web panel by Timoshenko and Gere [9] and after research on the shear buckling strength of a web panel based on the limited diagonal tension theory by Basler [10]. The post buckling strength of web panels was first established by Wilson [11] through a research conducted on the shear strength and behaviors of web panels. After Wagner [12] proposed the diagonal tension theory, much research has been carried out to evaluate the tension field action to explain the ultimate shear behavior and strength [13–16]. The research on the shear buckling strength has focused on the ultimate state of the stress after the shear buckling of the web panel. The results of Basler's research were adopted in the AASHOTO [17] and AISC [18] specifications.

The critical shear buckling load of a web panel with a rectangular section was determined using the buckling coefficient with regard to the boundary conditions [9], as shown in Eq. (1):

$$\tau_{cr} = k\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t_w}{h_w}\right)^2 \quad \text{for } A_f/A_w < 0.8 \tag{1}$$

where E is the elastic modulus, ν is the Poisson's ratio, t_w is the web thickness, h_w is the web height and k is the buckling coefficient, which depends on the boundary conditions and the aspect ratio.

In AASHTO design specifications [17], the nominal shear resistance (V_n) of the interior web panel is given by Eq. (2), on the basis of the fully plastic strength (V_p):

$$V_n = V_p\left(C_v + \frac{0.87(1-C_v)}{\sqrt{1+(d_0/h_w)^2}}\right) \tag{2}$$

where the fully plastic shear strength (V_p) and the shear coefficient (C_v) are given by Eqs. (3) and (4) below:

$$V_p = \tau_y h_w t_w = \frac{f_y}{\sqrt{3}} h_w t_w = 0.58 f_y h_w t_w \tag{3}$$

$$C_v = 1.0 \quad \text{for } \frac{h_w}{t_w} \le 1.12\sqrt{\frac{k_v E}{f_y}} \tag{4a}$$

$$C_v = 1.12\frac{\sqrt{k_v E/f_y}}{h_w/t_w} \quad \text{for } 1.12\frac{\sqrt{k_v E}}{f_y} < \frac{h_w}{t_w} \le 1.40\sqrt{\frac{k_v E}{f_y}} \tag{4b}$$

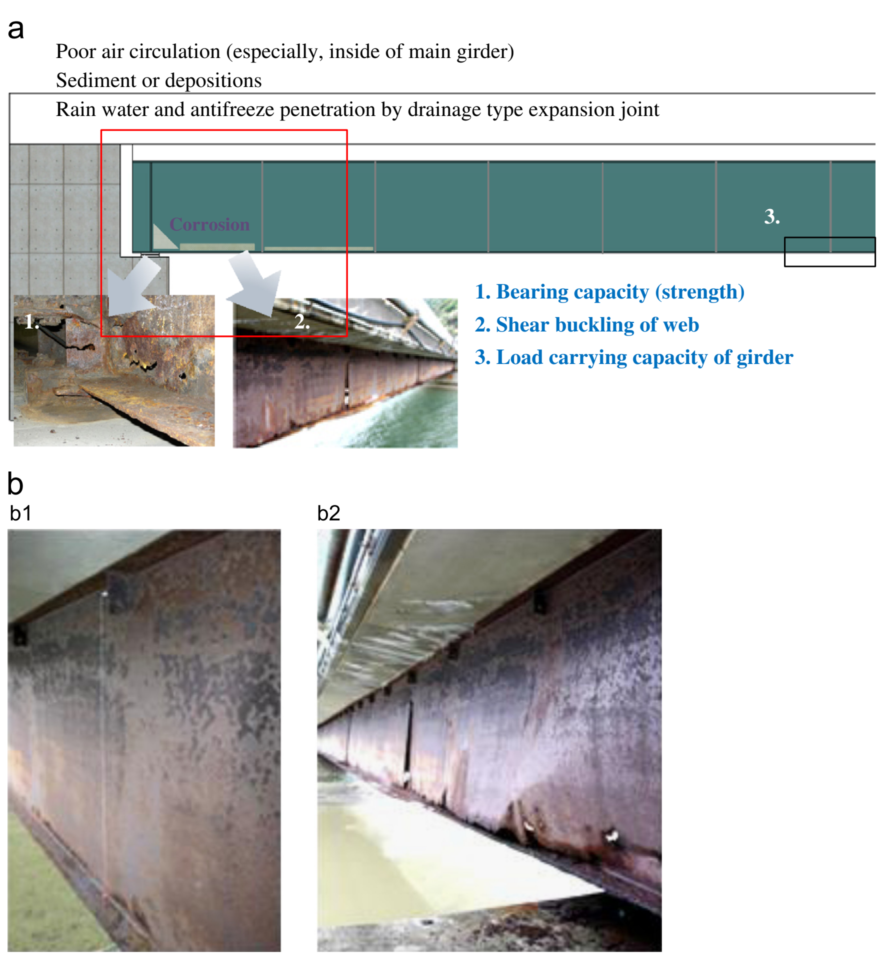

Fig. 1. Structural performances of plate girder bridges and the deformed corroded web girder bridge. (a) structural performances of plate girder bridges related to corrosion damages and (b) out of plane displacement of the corroded girder bridge depending on corroded periods, (b1) june, 2005, (b2) august, 2008.