METHOD EVALUATING LOCAL BUCKLING STRENGTH OF GUSSET PLATE CONNECTION WITH CROSS-SECTIONAL CORROSION SECTION

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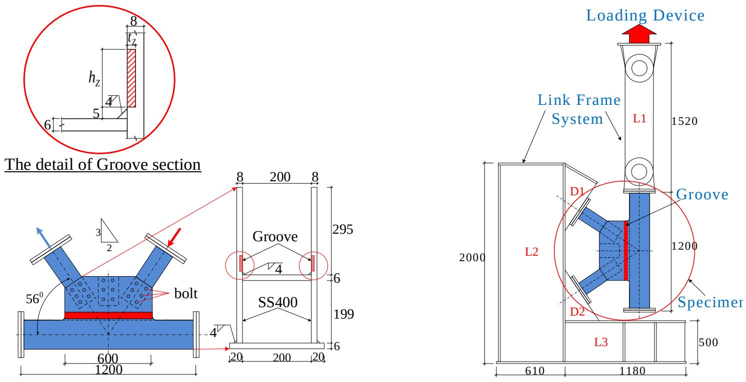
1. INTRODUCTION

In the world, the existing steel truss bridges are considered old because they are in the range of 50 to over 100 years, mostly have been built in the period 1950-1970 and before 1950. Further, numerous 100-year-old bridges still service now. With the considerable increase of the over-50-years steel truss bridges in the coming year, severe damage to the gusset plate connection of steel truss bridges owing to cross-sectional corrosion has become a serious problem, (see **Fig. 1**). Additionally, the reduction of the load-carrying capacity of the corroded gusset plate connection has been confirmed to can lead to the collapse of the entire truss bridge. Therefore, evaluation of the strength of the gusset plate connection, with the cross-sectional corrosion section on the gusset plate, has become a critical subject. In this study, based on the reached FEM parametric analysis result and the experimental result, the method evaluating the local buckling strength of the cross-sectional corrosion on the gusset plate was proposed.

Fig. 1 Frequently corroded location on gusset plate connection.



(a) Specimen with cross-sectional loss part (b) Link frame

Fig. 2 Specimen shape.

2. EVALUATION EQUATION IN LOCAL BUCKLING STRENGTH

2.1 Specimen shape

The specimens used in this study were of the monolith-type. These models were approximately 50% the size of the real bridge. The length and width of the gusset plate connection and the thickness of the gusset plate were 1200 mm, 216 mm, and 8 mm, respectively, as shown in **Fig. 2a**. Further, the cross-sectional loss part owing to corrosion was expressed by cutting a groove (called the “Groove”) at the location connecting the gusset plate and the upper flange of the lower chord member, with height h_z and width t_z. The loading test was carried out by the link frame system, as **Fig. 2b**.

2.2 Analysis model

Three-dimensional geometric nonlinear analysis was implemented for the gusset plate connections without and with the cross-sectional loss part on the gusset plate, with the displaced load type as shown in **Fig. 3**. The element type of the gusset plate connection and the loading members are the curved shell element, and the three-dimensional beam element, respectively. In the cases having the Groove section, only the Groove section is simulated in the solid brick element. The unit of finite element mesh in all of the models is 1 mm for Groove section, and 5 mm for the other members. Therefore, the total numbers of nodes and elements in the intact case and the cases with the cross-sectional loss part are 78223 and 27378, 101403 and 28476, respectively.

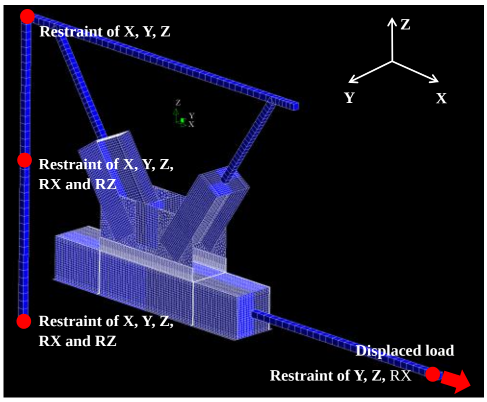


Fig. 3 Finite element analysis model.

2.3 Proposal of evaluation equation

This Section proposed the evaluation method of local buckling strength at the plate area underneath the compressive diagonal member, in the cases having the cross-sectional corrosion. For the specimens in the experimental environment, the diagonal members of the gusset plate connection were connected to the gusset plate by using bolts through the connecting plates. Therefore, the effective width of the buckling plate area was determined in accordance with the Whitmore method[1)] (see **Fig. 4a**). The local buckling strength of the gusset plate with the cross-sectional corrosion, was calculated as the column with the sudden-changed cross section, and fixed ends as shown in **Fig. 4b**. The buckling load condition of the column with sudden change in cross section was determined as Equation (1).

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$$\left(\frac{\lambda_1}{\lambda_2}\right)\tan(\lambda_1 l_1)+\tan(\lambda_2 l_2)=0, \quad \text{(1a)}$$

$$\lambda_1=\sqrt{P/EI_1}, \quad \text{and} \quad \lambda_2=\sqrt{P/EI_2} \quad \text{(1b)}$$

Where

l_1, l_2: The length of the column (see **Fig. 4b**)

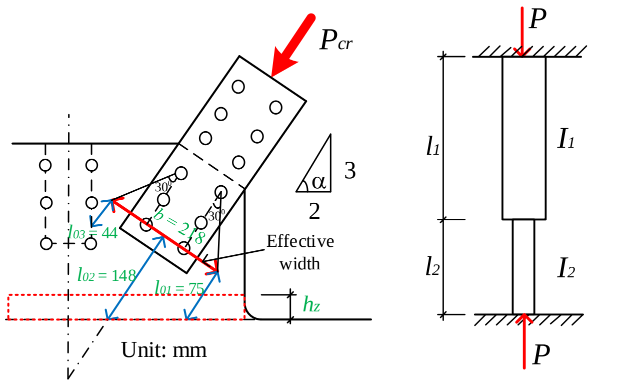
I_1, I_2: The moment of inertia of the column (see **Fig. 4b**)

The process calculating the local buckling strength of the compressive plate area was described as follows.

Firstly, the slenderness ratio of the plate area occurred local buckling was determined as the average value of three the component slenderness ratios, which were calculated in each the component column having the length from the effective width of the diagonal member to the upper flange of the lower chord member (l_{01}, l_{02}, and l_{03}).

$$\begin{cases} l_{01}\Rightarrow \bar{l}_{01}=\pi\sqrt{\dfrac{EI}{P_1}} \\ l_{02}\Rightarrow \bar{l}_{02}=\pi\sqrt{\dfrac{EI}{P_2}} \\ l_{03}\Rightarrow \bar{l}_{03}=\pi\sqrt{\dfrac{EI}{P_3}} \end{cases} \Rightarrow \begin{cases} \lambda_{c1}=\left(\dfrac{1}{\pi}\right)\sqrt{\dfrac{\sigma_y}{E}}\left(\dfrac{\bar{l}_{01}}{r_s}\right) \\ \lambda_{c2}=\left(\dfrac{1}{\pi}\right)\sqrt{\dfrac{\sigma_y}{E}}\left(\dfrac{\bar{l}_{02}}{r_s}\right) \\ \lambda_{c3}=\left(\dfrac{1}{\pi}\right)\sqrt{\dfrac{\sigma_y}{E}}\left(\dfrac{\bar{l}_{03}}{r_s}\right) \end{cases} \quad \text{(4a)}$$

$$\Rightarrow \bar{\lambda}_c=\frac{\lambda_{c1}+\lambda_{c2}+\lambda_{c3}}{3} \quad \text{(4b)}$$



(a) Local buckling plate area b) The column with fixed ends

Fig. 4 Local buckling underneath the compressive diagonal.

Where

l_{01}, l_{02} and l_{03} : The lengths of three the component columns. σ_y : Yield stress of steel.

P_1, P_2 and P_3 : The local buckling strength in each the component column.

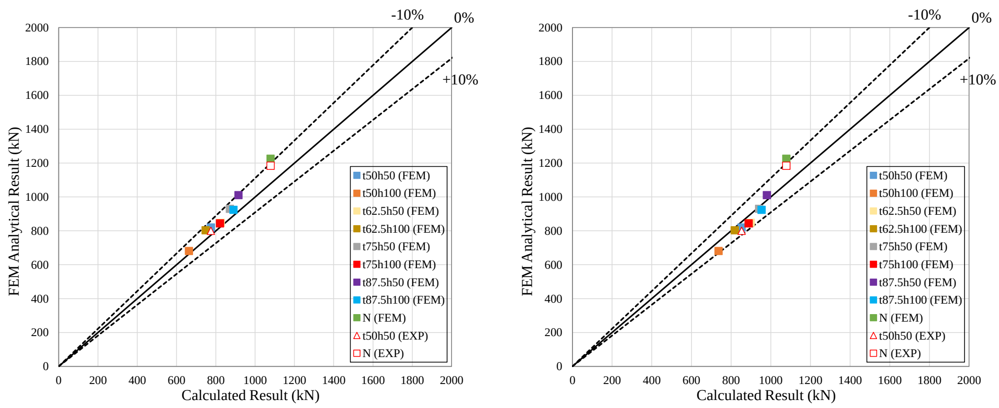
$\bar{l}_{01}$, $\bar{l}_{02}$ and $\bar{l}_{03}$: The effective buckling lengths of three the component columns. r_s : Radius of gyration.

λ_{c1}, λ_{c2} and λ_{c3} : The slenderness ratio of each the component column.

Finally, the local buckling strength of the compressive plate area was determined by using the standard buckling equations specified in Japanese Design code (JSHB) with the calculated average slenderness ratio value.

2.4 Calculated result and discussion

In this Section, to confirm the accuracy of the proposed evaluation method, the FEM parametric analyses were implemented by changing the dimension of the cross-sectional loss part on the gusset plate. The relation of the local buckling strength between the calculated result (CAL), the FEM analytical result and the experimental result (EXP) is shown in **Fig. 5a** for comparison. Particularly, to evaluate the difference of the calculated result when using another standard buckling strength curve, the result calculated by using AASHTO was added, as shown in **Fig. 5b**. From **Fig. 5**, when using the buckling strength curve of JSHB; it is confirmed that the difference of the strength value between CAL, FEM and EXP was in the range of -10% to 0% on the safe side, and obtained the calculated result with the high level of accuracy. Conversely, when using the buckling strength curve of AASHTO, this difference was in the range of -10% to +10% with a part on the safe side. This is understood that with the same slenderness ratio value the buckling strength under the strength curve of JSHB is usually lower than that of AASHTO. Therefore, to evaluate safely the local buckling strength of the compressive plate area, using the standard buckling strength curve of JSHB was strongly considered.



(a) Calculated by using JSHB (b) Calculated by using AASHTO

Fig. 5 The local buckling strength from CAL, FEM and EXP.

3. CONCLUSIONS

In this study, the local buckling strength of the gusset plate connection with the cross-section corrosion on the gusset plate was proposed to calculate as the column having the sudden-changed cross section and be fixed in two the ends. As a result, it is confirmed that the calculated result obtained from the proposed evaluation method was on the safe side in the range of -10% to 0%, compared to the FEM analytical result and the experimental result.

REFERENCES

Whitmore, R. E.: Experimental Investigation of Stresses in Gusset-Plates, University of Tennessee, Engineering Experiment Station, Bulletin, No. 16, May, 1952.