ACCURACY EVALUATION ON FE ANALYSIS OF WELDED BOX-SECTION COLUMNS UNDER AXIAL COMPRESSION

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1. Introduction

Recently, an analytical study[1)] on coupled buckling strength of steel compression members with box section was conducted. However, the accuracy of FE analysis was not sufficiently evaluated. In this study, a series of FE analysis of the specimens tested by Usami et al.[2)] is carried out to investigate their load bearing capacity. Then FE analysis results are compared with test results on ultimate strength in order to evaluate the accuracy of FE analysis.

2. FE Analysis

FE analyses of 27 models in Usami's test, in which initial deflection and residual stress are considered, are performed by MSC.Marc. Table 1 shows slenderness (λ) and width-thickness ratio (b/t) of the specimens. Coupled buckling is expected to occur in the specimens with the λ-value larger than 35 and b/t-value larger than 27. The symbols of S, R and ER represent square column under central loading, rectangular column under central loading and rectangular column under eccentric loading conditions, respectively.

2.1 Element and material properties

Thick shell element 75 is used to develop the finite element model. Every plate is divided into 20 elements in horizontal direction so that residual stress can be easily introduced. According to the actual measured stress-stain diagram, properties of material are determined as yield stress is 741N/mm^2, Young's modulus is 215000N/mm^2, stain at yield stress is 0.549%, stain hardening modulus is 2690N/mm^2 and Poisson's ratio is 0.24.

2.2 Boundary condition

One node is set at midpoint of the top cross section and bottom cross section, respectively. The central node is set as the main node and the other nodes in the cross section are set as subordinate nodes. Then the function "RBE2" is used to connect the central node with the whole nodes at the cross section so that the cross section becomes rigid section just like a steel plate with very large stiffness. Fig.1 shows the boundary conditions of the models. It can be seen that only rotation Y is free at the bottom point. At the top point, four degrees of freedom are fixed except rotation Y and displacement Z. In order to get ultimate strength accurately, displacement loading is used in the analyses. And the loading is applied to the top main node.

2.3 Initial deflection

Initial deflection includes overall deflection along the length of the specimen w_g and local deflection on the plate w_l. For the global deflection, measured maximum deflection with a half sinusoidal wave shape expressed by Eq. (1) is assumed along the length of the columns. On the other hand, double trigonometric function shown in Eqs. (2) and (3) is assumed for local deflection.

Table 1 Slenderness and width-thickness ratio of specimens

λ \ b/t	22	27	33	38	44
10	S-10-22 R-10-22	S-10-27	S-10-33 R-10-33	S-10-38	S-10-44 R-10-44
35	S-35-22	S-35-27	S-35-33	S-35-38	S-35-44
50	S-50-22 R-50-22 ER-50-22	S-50-27 R-50-27 ER-50-27	S-50-33 R-50-33 ER-50-33	R-50-38	R-50-44
65	R-65-22	R-65-27	R-65-33	–	–

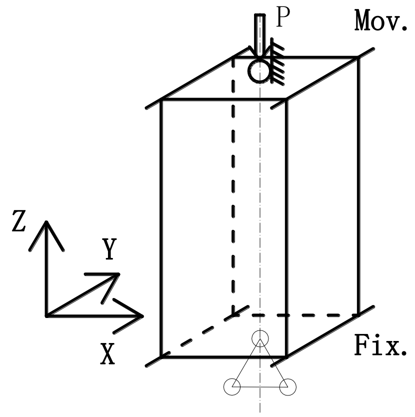

Fig. 1 Boundary condition

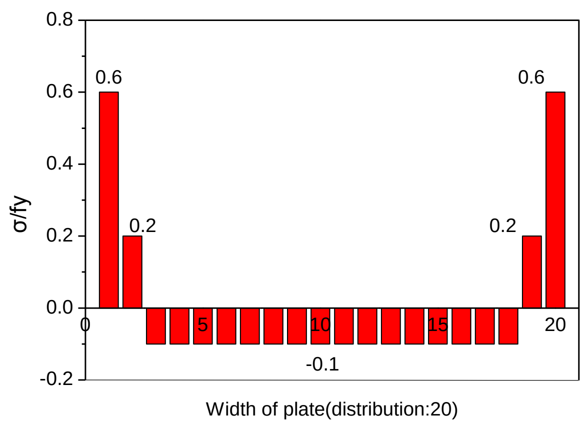

Fig. 2 Distribution of residual stress

$$w_g = A\sin(\frac{\pi z}{L}) \quad (1)$$

Where, A is for actual maximum value of measured deflection and L is the length of columns.

$$w_l = w_{l0} \sin \frac{m\pi z}{L} \cos \frac{\pi x}{b} \quad (2)$$

Or $$w_l = w_{l0} \sin \frac{m\pi z}{L} \cos \frac{\pi y}{b} \quad (3)$$

Where, m is the number of half sinusoidal wave giving the minimum buckling strength determined by aspect ratio of the plate, and b is the plate width. The maximum value of local deflection w_{l0} is assumed as $b/1000$ although it is not measured in the test.

2.4 Residual stress

According to the residual stress measurement in the test, welded area between flange and web possesses tensile residual stress approximately equal to 0.6 of yield stress. Mid area of the plate possesses compressive residual stress approximately equal to 0.1 of yield stress. Therefore, the residual stress distribution in a plate is assumed as shown in Fig. 2 so as to be in self-equilibrium condition.

2.5 Model solution

Since this study focuses on the ultimate strength of welded box-section steel columns under axial compression, both of material nonlinearity and geometrical nonlinearity need to be considered. The material nonlinearity is considered based on the measured stress-stain diagram. As for the geometrical nonlinearity, the function of “large stain” is on. In addition, arc length method is introduced to solve the nonlinear equation, in which arc length serves as increment and convergence will be reached along the arc length. It can lead to more accurate solution. And initial displacement is set to a value small enough to prevent from overlarge convergence ratio at the first loading step.

3. Comparison and evaluation

Ultimate strength obtained by FE analysis is compared with that of test. Comparisons are shown in Fig.3, Fig.4 and Table 2. It can be seen that most of the FE analysis results are close to the test results with errors less than 5%. For the square columns, errors between test and FE analysis vary from -5.11% to 7.26%. Average of errors is 2.32%. The standard deviation (SD) is 4.02%. For the rectangular columns, errors between test and FE analysis vary from -7.12% to 7.78%. Average of errors is 1.32%. The standard deviation (SD) is 4.99%. Consequently, it can be concluded that the FE analysis has sufficient accuracy.

4. Conclusion

Main conclusions of this study can be summarized as follows.

(1) By using arc length method, accurate nonlinear solution can be obtained.

(2) FE models considering actual initial deflection and residual stress will achieve accurate prediction of ultimate strength of high strength steel welded box-section columns.

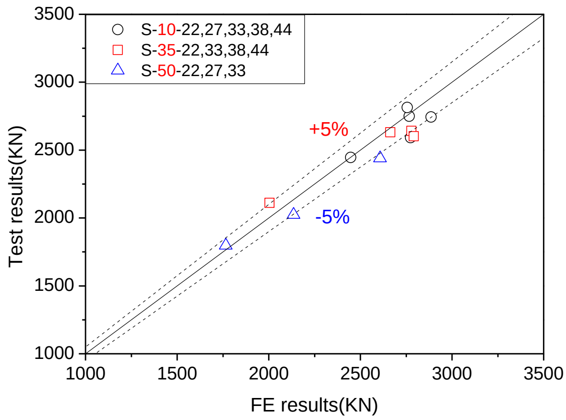

Fig. 3 Comparison of ultimate strength for square columns

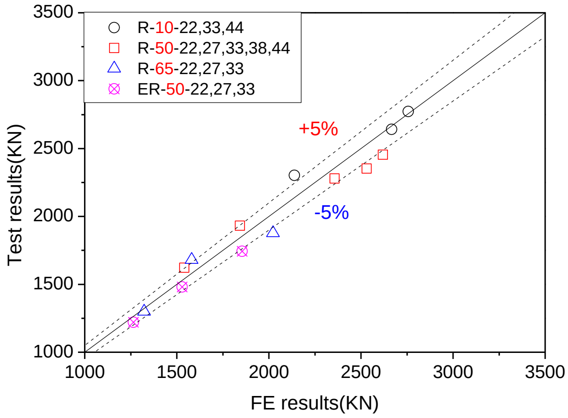

Fig. 4 Comparison of ultimate strength for rectangular columns

Table 2 Comparison of ultimate strength between FE analysis and test

Specimen	Errors(%)	Specimen	Errors(%)
S-10-22	0.08	R-10-22	-7.12
S-10-27	7.02	R-10-33	0.98
S-10-33	0.65	R-10-44	-0.58
S-10-38	5.17	R-50-22	-4.99
S-10-44	-2.03	R-50-27	-4.56
S-35-22	-5.11	R-50-33	3.42
S-35-27	—	R-50-38	7.61
S-35-33	5.23	R-50-44	6.68
S-35-38	7.26	R-65-22	1.85
S-35-44	1.22	R-65-27	-5.95
S-50-22	-1.78	R-65-33	7.78
S-50-27	5.53	ER-50-22	3.69
S-50-33	6.89	ER-50-27	3.31
—	—	ER-50-33	6.36
Average	2.32	Average	1.32
SD	4.02	SD	4.99

References

1) Yusuke Kishi et al.: Numerical evaluation for coupled buckling strength of steel compression members with box section, Proceedings of EASEC-14, pp.1889-1896, 2016.

2) Tsutomu Usami et al.: Local and Overall Buckling of Welded Box Columns. Journal of the Structural Division, ASCE, 108(ST3), pp.525-542, 1982.