NUMERICAL INVESTIGATION ON FLEXURAL BUCKLING STRENGTH OF H-SECTION COLUMNS CONSISTING OF ULTRA-HIGH STRENGTH STEEL FLANGE AND NORMAL STRENGTH STEEL WEB

Shotaro Hayakashi[1], **Hayato ASADA**[2], and **Kenzo TAGA**[3]

SUMMARY

Hybrid WF cross-section steel (HWF) column which consists of ultra-high strength steel flange with a nominal yield strength of 810MPa and normal strength steel web, has been proposed to take the structural advantages of ultra-high strength steel, reducing the fabrication costs. This paper numerically investigates the flexural buckling behavior of HWF columns. A non-linear finite-element model (FE model) was developed for HWF columns. The parametric study was conducted comprising 180 FE models to investigate the effect of difference in strength between the flange and the web and thickness of the web on the flexural buckling strength of HWF columns. The large strength difference between flange and web results in web yielding prior to flexural buckling, but even in the inelastic buckling range, the effect of prior yielding of the web on the reduction in flexural buckling strength is not significant. The numerical results were compared to the current design equations per the US and Japanese design specifications. It was found that the flexural buckling strength of HWF column can be evaluated to be equivalent to that of conventional H-section columns by using the equivalent yield strength of the web and flange.

Keywords: *Ultra High-Strength Steel; H-section Column; Flexural Buckling Strength; FE analysis.*

INTRODUCTION

High-strength steel (HSS), which has higher strength and yield ratio compared to the conventional mild steel, has become applied to building structures (e.g., Sasaki et al. 2011), in the high seismic regions such as Japan and China. It is widely realized that the proper utilization of HSS in seismic structural engineering provides many essential advantages such as economy and environment-friendly. BT-HT880 with a nominal yield strength of 810MPa (Kawabata et al. 2011) as well as H-SA700 (Yoshida et al. 2009) steel with a nominal yield strength of 700MPa, which are manufactured using the thermo-mechanical control process (TMCP) to achieve very high strength and good weldability neither significantly increasing alloying elements nor using intensive heat treatment, have been developed and commercialized in Japan (Xiang et al. 2016).

With the aims of effectively using these ultra-HSSs, extensive studies have been made on the members that were made of newly HSSs as well as the relevant joint connection method (e.g., Tsuda et al. 2011). However, in order to further promote the utilization of HSS, it is of important concern to reduce the fabrication costs, especially for the welding. Although newly HSSs have good weldability, the welding using matched or even wire electrodes with special treatments (i.e., pre-heat or post-heat treatments) is required at the current stage, which results in cost performance worse than conventional structural steel. In order to overcome this difficulty, in this study, a built-up hybrid WF (HWF) cross-section which consists of flange made of HSS, and web made of conventional structural steel assembled by the welding using a conventional electrode without any special treatment was proposed (Kato et al. 2017).

[1] *Graduate student, Shibaura Institute of Technology, Japan, e-mail:mj22108@shibaura-it.ac.jp*
[2] *Associate Professor, Shibaura Institute of Technology, Japan, e-mail: hasada@shibaura-it.ac.jp*
[3] *Director, Structure Judgement center, General Building Research Corporation of Japan, Japan, e-mail: k-taga@gbrc.or.jp*

The HWF cross-section is intended to utilize for the column in building structure with damping system which make it possible to continuous use of the buildings even after severe earthquake. Taga et al. suggested that the elastic deformation capacity of primary structures, i.e. column and beam, improved by the utilization of HSSs with the certain stiffness, can reduce the maximum drift of buildings on the premise of damped system, under sever pulse-type earthquake beyond design-basis (Taga et al. 2012). Incremental dynamic analysis (IDA) conducted by Lin have revealed the utilization of HSS for the column in braced frame buildings lead to reduce both maximum and residual drifts (Lin. 2012).

Because of the large strength difference between the web and flange in HWF columns, there is concern that shear yielding of the web will limit strength in the practical range of column size and lengths. In order to prevent premature shear yielding of the web while allowing axial yielding, the plastic strength of the HWF cross-section for axial, bending, and shear was derived and considered in the design. In addition, static load tests and associated finite element analyses has been conducted primarily to investigate the quasi-elastic deformation capacity, ultimate strength, and post-buckling resistance behavior of HWF beam-columns (Asada et al. 2020).
For column design, it is also important to evaluate the load-carrying capacity determined by the flexural buckling of the member. For this reason, this paper numerically investigates the flexural buckling behavior of HWF columns in the practical range of column length, primarily focusing on the combination of the strength in the flange and web and thickness of the web.

NUMERICAL INVESTIGATION

Description of the FE model and parameters

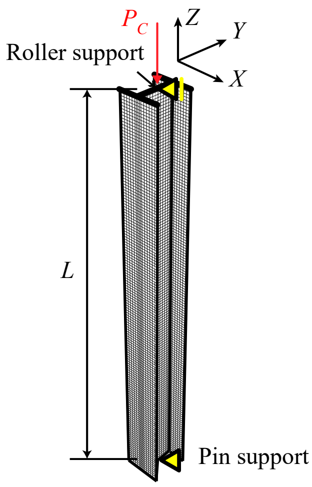
In order to investigate the flexural buckling strength of HWF columns and examine the applicability of current design equations, the FE models are developed using the finite element software Abaqus ver. 6.21. Fig.1 shows an example of the FE model and selected cross-section shapes. The FE models were simply-supported columns subjected to axial compression and were modeled by a 3D shell element (S4R). To study both strong- and weak-axes buckling behavior, the corresponding boundary condition was adopted. Two cross-sections, H-300×300×12×19 and H-300×300×32×19 are selected. For HWF columns, as previously mentioned, because of the large difference in strength between the flange and web, HWF column can occur shear yielding of the web prior to flexural yielding of flange even at realistic shear span ratios. Thus a cross-section with a thicker web than a typical H-beam, 32 mm, was prepared.
Note that even for HWF columns, the width-to-thickness ratio of the web and the flange are determined to satisfy at least the limitation specified in the allowable stress design criteria of AIJ, which are given by the following equation.

$$^{b}/_{t_f} \leq 0.53\sqrt{^{E}/_{\sigma_{fy}}} \tag{1}$$

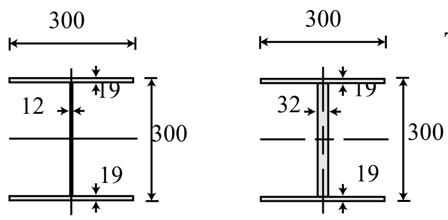
where b is half of flange width, t_f is flange thickness, E is Young's modulus.
Other than the thickness of the web, the combination of the material strength in the flange and the web were considered as shown in Table 1. two steel materials were used: BT-HT880, an ultra-high strength steel with a yield stress of 950 MPa, and SN490B, an ordinary steel with a yield stress of 358MPa. The yield stresses for both steels were determined based on past experimental results, not nominal values (Endo et al. 2013). In order to compare the buckling behavior between the conventional columns and the proposed HWF columns, two combinations of the materials were employed: one in which the ultra-high strength steel was used only for the flange, and the other in which ordinary steel was used for both the flange and the web.
For the investigation, the global slenderness ratio λ is varied within the practical range by varying the column length L, as shown in Table 2.

Loading method and Imperfection

The linear buckling loads and buckling modes were obtained by eigenvalue buckling analysis. Then buckling mode corresponding to the flexural buckling (overall buckling) mode, that result from an eigenvalue buckling analysis, was imposed as an initial imperfection where, the maximum imperfection amplitude was 1/1000 L. A finite displacement analysis was then performed. the constitutive equations for the plastic region in the finite displacement analysis follow the von Mises yield criterion, the associated flow law, and the isotropic hardening law. The stress-strain relationship of the steels was assumed to be linearly elastic to perfectly plastic. No residual stress was assigned since there are no residual stress measuring tests for the HWF columns.



(a) **FE model**



(b) Cross-section 1 (c) Cross-section 2

Fig.1 FE model and cross-sectional dimensions

Table.1 Cross-section of FE model and combination of steels used

Analysis Model	Cross-section	Yield Stress (N/mm^2)		Equivalent Yield Stress σ_{ye} (N/mm^2)
		Flange σ_{fy}	Web σ_{wy}	
HWF-1	1	960	358	830
CWF-1	1	358	358	358
HWF-2	2	960	358	705
CWF-2	2	358	358	358

Table.2 Ranges of column length and slenderness ratio

Length L (mm)	500~25,000
Slenderness Ratio λ	3.9~204

NUMERICAL RESULTS

Load – Deformation responses

Average compressive stress versus average compressive strain along the column relationships are shown in Fig.2. These figures show the results of HWF and CWF columns for λ=21.3~130 in both axes. Average compressive stress and average compressive strain are determined by:

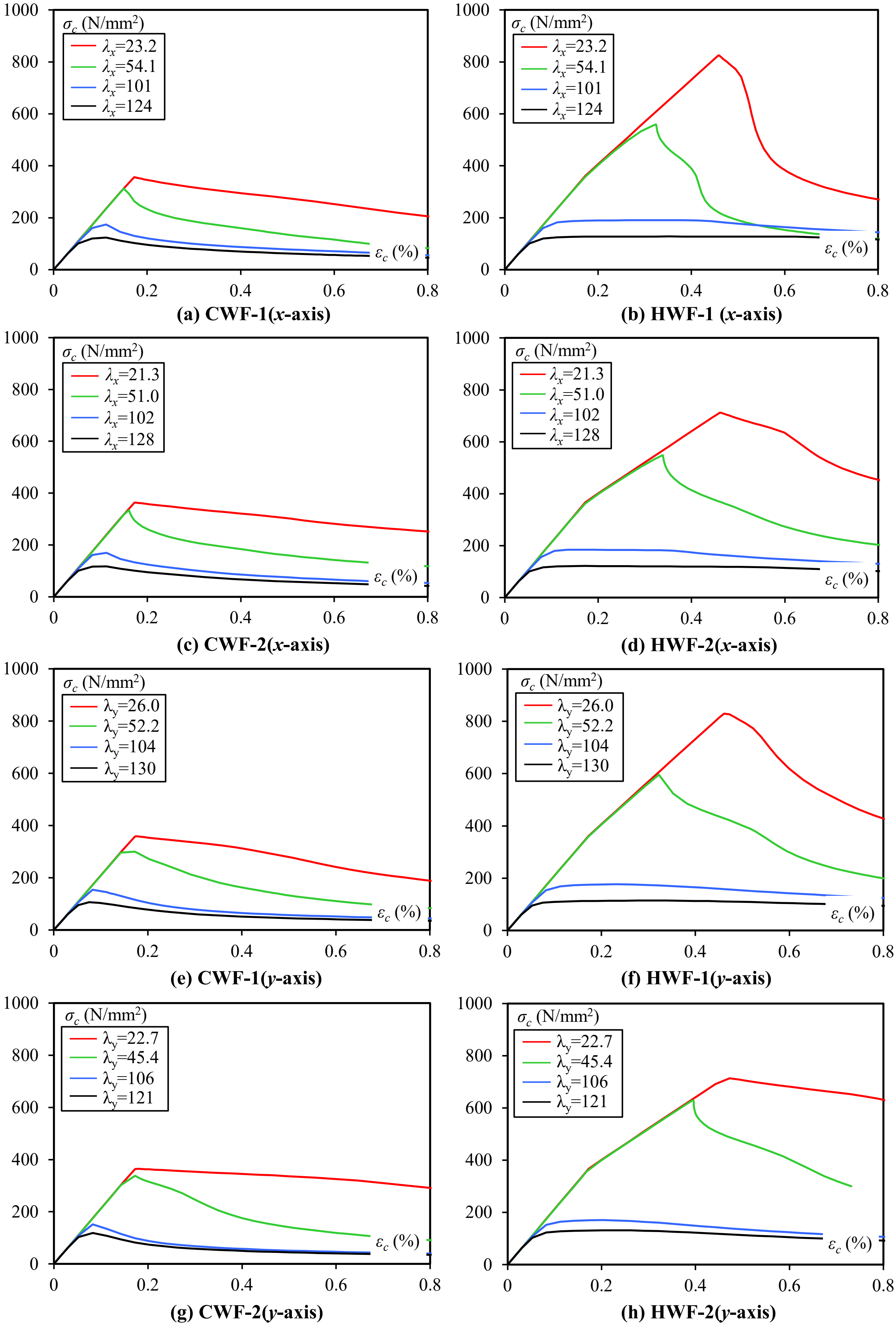
$$\sigma_c = \frac{P_c}{A} \quad (2)$$

$$\varepsilon_c = \Delta L / L \quad (3)$$

where P_c and ΔL are axial compressive load and axial displacement respectively.

When the column have the slenderness λ in elastic buckling range, there are no differences in the flexural buckling strength of CWF columns, with both flange and web made of normal strength steel and HWF columns with only flange made of HSS, when compared with the same slenderness ratio. However, in inelastic buckling range, the flexural buckling strength of the HWF columns is greater than those of the CWF columns. In addition to this, although the compressive strain at the buckling strength is about 0.2% or less for CWF columns, it is 0.4% or more for HWF columns at $\lambda \leqq 30$. Therefore, HWF columns can attain a larger ultimate deformation determined by flexural buckling compared to conventional WF columns.

Looking at the results with respect to the column cross-sectional shapes, In CWF columns, different web thicknesses do not cause significant differences in buckling strength and post-buckling behavior. On the other hand, in HWF columns, HWF-2 with a thicker web has less degradation in the strength after buckling. It could be because a thicker web can make the HSS flanges more constrained and minimize the loss of bearing capacity due to local buckling that occurs in the center of the member after flexural buckling has occurred. It can be said that the increase in web thickness of HWF columns provides advantages of not only prevention of premature shear yielding of the web but also in reducing degradation behavior after buckling.



(a) CWF-1(*x*-axis) **(b) HWF-1 (*x*-axis)**

(c) CWF-2(*x*-axis) **(d) HWF-2(*x*-axis)**

(e) CWF-1(*y*-axis) **(f) HWF-1(*y*-axis)**

(g) CWF-2(*y*-axis) **(h) HWF-2(*y*-axis)**

Fig.2 Average compressive stress-compressive strain relationships

Effects of premature axial yielding of the web

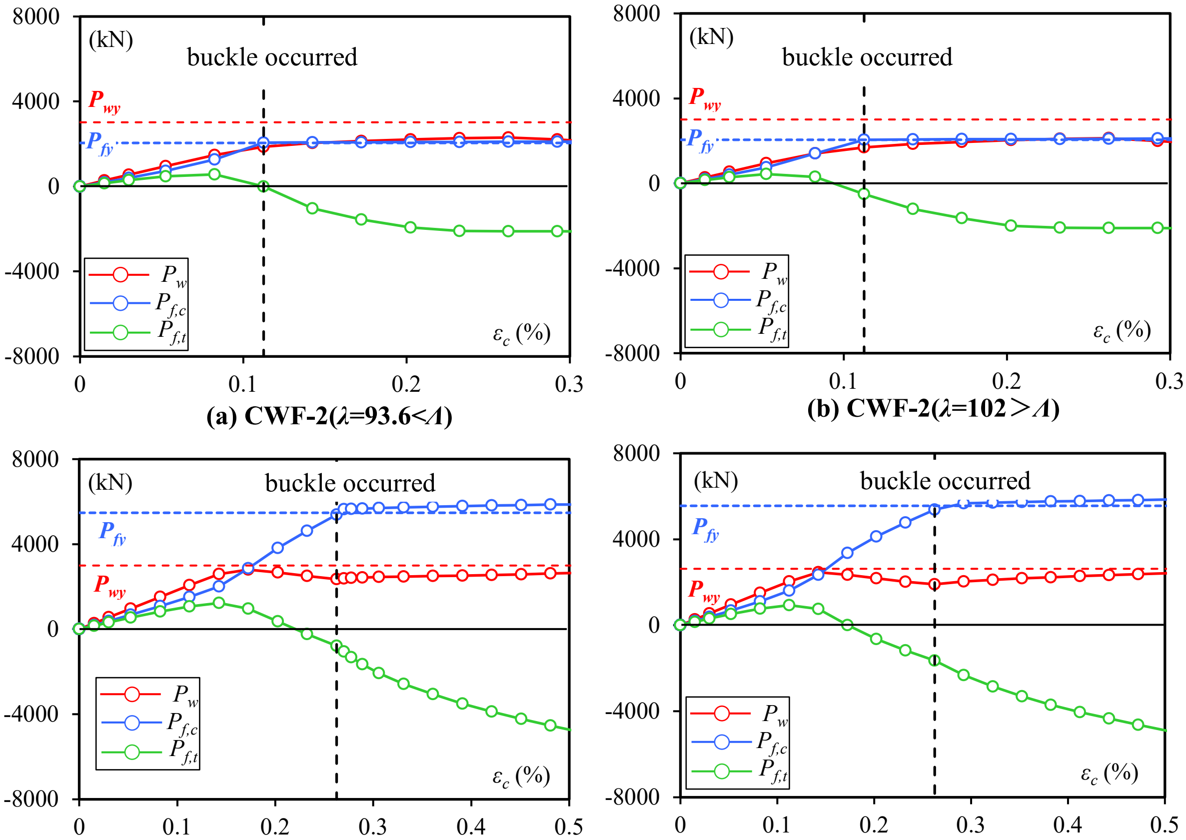
In the case of HWF columns, the strength of the web is relatively lower than that of the flange, therefore there is a possibility that the web will yield before reaching the buckling capacity. Focusing on CWF and HWF columns with cross-section 2 (i.e., CWF-2 and HWF-2), axial forces carried by the flanges P_{ft} (tension side) and P_{fc} (compression side) and that carried by the web for the models with λ that are around the transition points elastic to inelastic buckling range Λ are shown in Fig.3. Λ is the transition point from elastic to inelastic buckling ranges defined by the following equation, according to AIJ allowable stress design specification.

$$\Lambda = \pi\sqrt{E/_{0.6}\sigma_{ye}} \quad (4)$$

where, E is Young's modulus, σ_{ye} is the equivalent yield stress.
Note that Λ is obtained here using the equivalent yield stress defined by the following equation:

$$\sigma_{ye} = \frac{\sigma_{wy}A_w + 2\sigma_{fy}A_f}{A} \quad (5)$$

where σ_{wy} and σ_{fy} are yield stresses of web and flange respectively, A_w and A_f were cross-sectional area of the web and the flange respectively, A is gross sectional area.
For CWF columns, the magnitude of the axial force carried by the web remains below the yield axial force P_w when the buckling capacity is reached, whereas for HWF columns, the axial force of the web reaches the yield axial force P_{wy} before buckling occurs. For HWF columns, therefore, the axial yielding of the web, which occurs before buckling, should be taken into account when determining the design capacity. However, the fact that the premature yielding of the web is not a factor that causes the buckling load to plateau. In addition, the reduction in bending stiffness of columns due to axial yielding of the web is not so significant for WF cross-section. Furthermore, when specifying the design bearing capacity, it is noteworthy that the web can bear an axial force equivalent to the yield axial force when the buckling capacity is reached.



(a) CWF-2(λ=93.6<Λ) **(b) CWF-2(λ=102>Λ)**

(c) HWF-2(λ=68.1<Λ) **(d) HWF-2(λ=72.3>Λ)**

Fig.3 Axial forces transmitted by flanges and web

Comparison with the current design equations

Figs 4 and 5 show comparisons of the numerical result and Japanese design equations (AIJ design equation) per AIJ Design Standard for Allowable Stress Design of Steel Structures (AIJ, 2021) and American design equations per the Specification for Structural Steel Building (AISC, 2015) . AIJ design equations are given by the following equations.

$$\sigma_{cr} = \frac{1.5\pi^2 E}{\lambda^2\left({}^{13}/_{6}\right)} \quad for\ \lambda > \Lambda = \pi\sqrt{\frac{E}{0.6\sigma_y}} \tag{6}$$

$$\sigma_{cr} = 1.5\frac{\left\{1-0.4\left(\frac{\lambda}{\Lambda}\right)^2\right\}\sigma_y}{\frac{3}{2}+\frac{2}{3}\left(\frac{\lambda}{\Lambda}\right)^2} \quad for\ \lambda < \Lambda \tag{7}$$

where, σ_y is yield stress.

American design equations are given by the following equations

$$\sigma_{cr} = \left(0.658^{\frac{\sigma_y}{F_e}}\right)\sigma_y \text{ for } \lambda \leq 4.71\sqrt{\frac{E}{\sigma_y}} \tag{8}$$

$$\sigma_{cr} = 0.877F_e \text{ for } \lambda > 4.71\sqrt{\frac{E}{\sigma_y}} \tag{9}$$

where, F_e: elastic buckling stress (Euler's flexural buckling stress).

Three column curves which are calculated using the yield stress σ_{fy} (960 N/mm2) of the flange, σ_{wy} (358 N/mm2) of the web, and the equivalent yield stress σ_{ye} are shown in the figure. For HWF columns, the design values based on σ_{wy} greatly underestimates the analytical results in the inelastic buckling region, while those based on σ_{fy} tends to overestimate the analytical results in the inelastic buckling region. On the other hand, the design values based on the equivalent yield stress σ_{ye} show the best and safest evaluation of the analytical results for the HWF column.

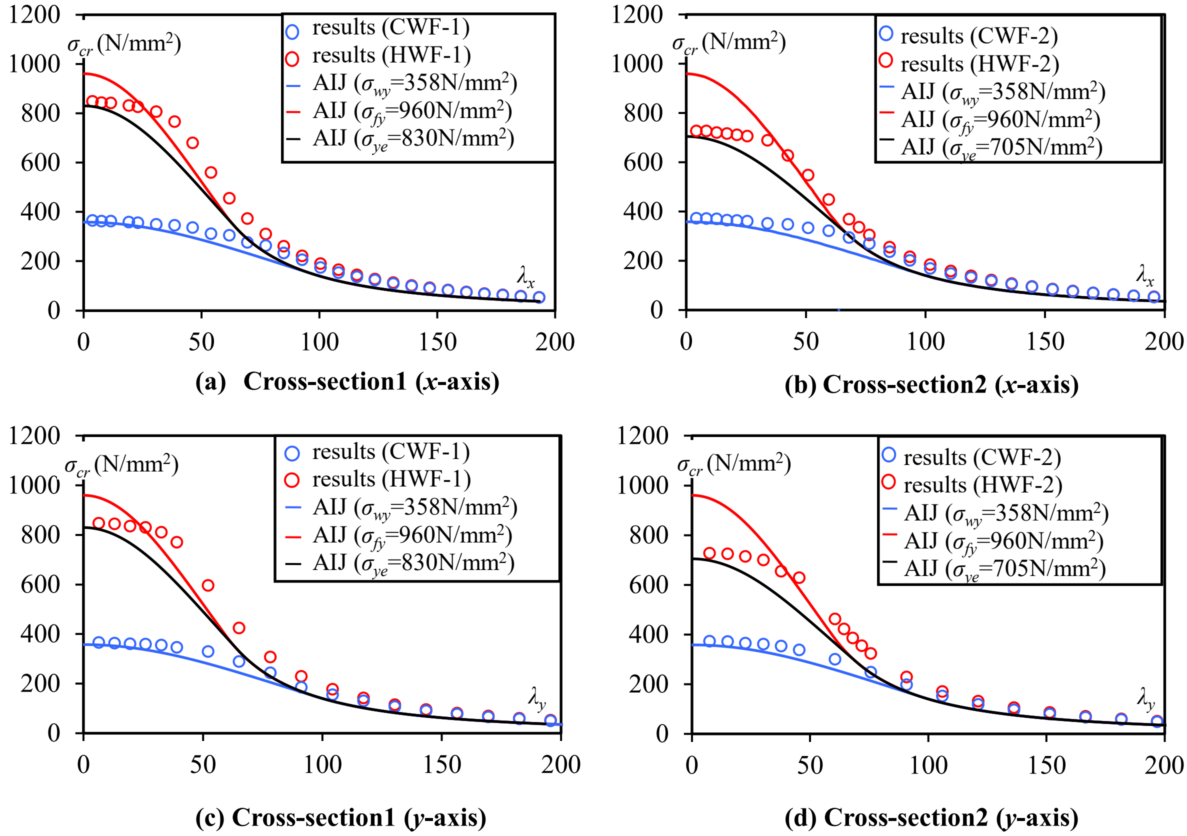
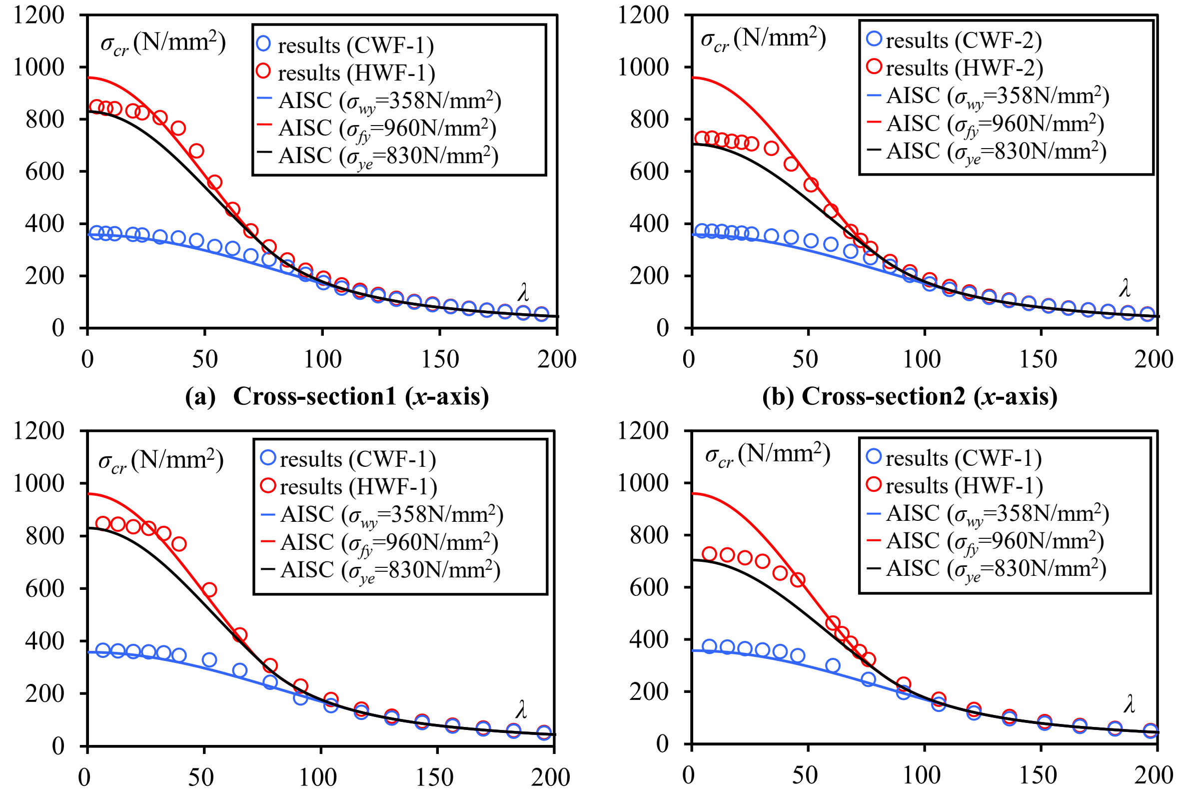


Fig.4 Comparisons of AIJ column curves with numerical results



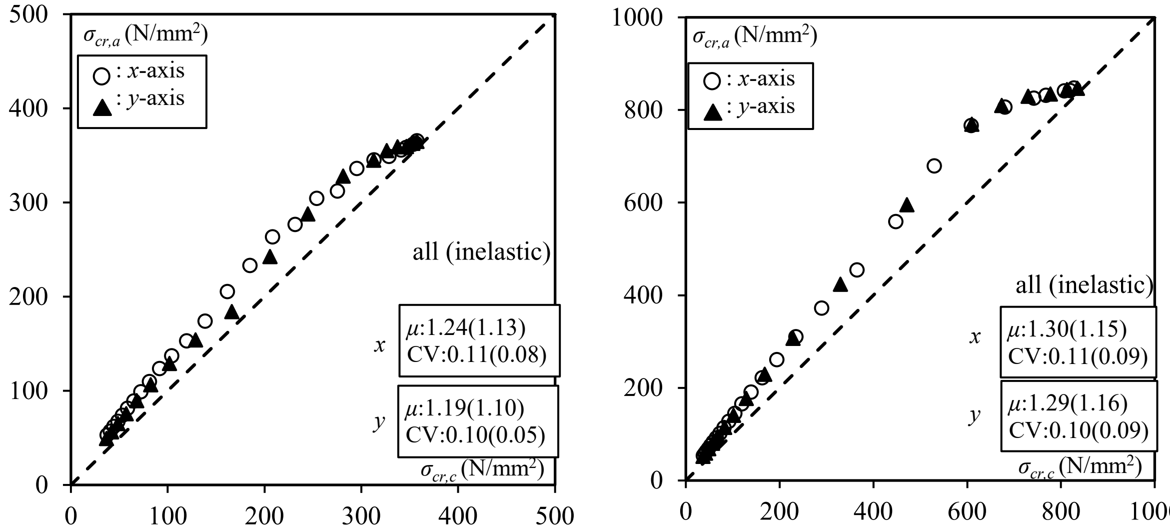
(a) Cross-section1 (*x*-axis) **(b) Cross-section2 (*x*-axis)**

(c) Cross-section1 (*y*-axis) **(d) Cross-section2 (*y*-axis)**

Fig.5 Comparisons of AISC column curve with numerical results

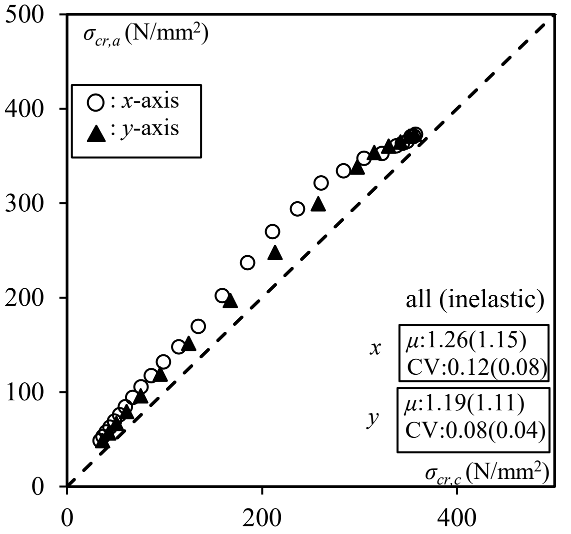
Validation of the use of equivalent yield stress

Figs 6 and 7 show comparison of numerical results $\sigma_{cr,a}$ and design buckling stress $\sigma_{cr,c}$. Regarding comparison with AIJ, the average values of the ratio between the numerical results to design strength μ range from 1.19 to 1.26 for CWF columns and are1.29 to 1.30 for HWF columns. While the coefficient of variation COV are 0.08 to 0.12 for both CWF columns and HWF columns. In comparison with AISC, μ are 1.04 to 1.10 for CWF columns and 1.10 for HWF columns. COV range from 0.035 to 0.04 for CWF columns and 0.033 to 0.04 for HWF columns. Although AIJ design equation tends to slightly underestimate the analytical results compared to AISC, both equations use σ_{ye} to evaluate the buckling strength of HWF columns as equivalent to that of CWF columns regardless of the cross-sectional shape and buckling axis.

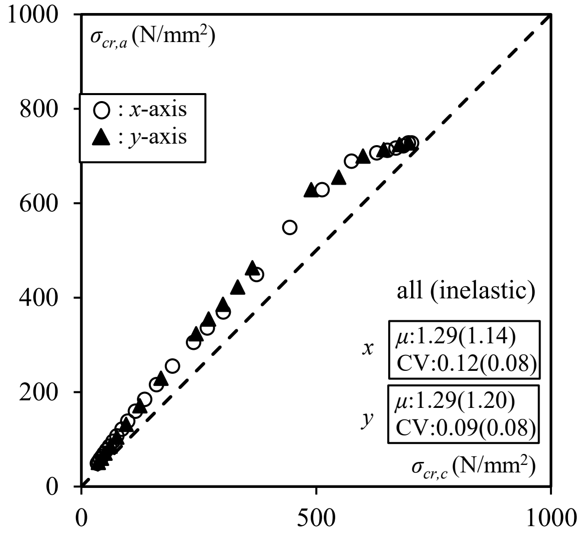


(a) CWF-1 **(b) HWF-1**

Fig.6 Comparison of analytical and calculated buckling stress values(AIJ)

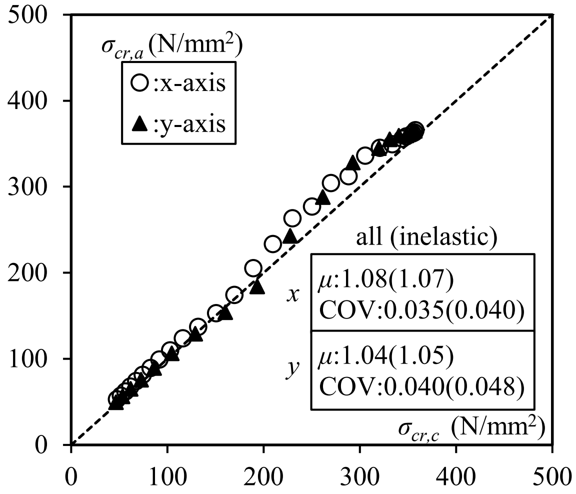


(c) CWF-2

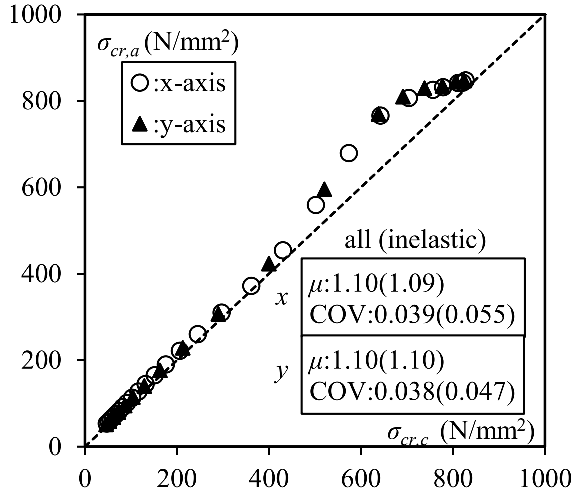


(d) HWF-2

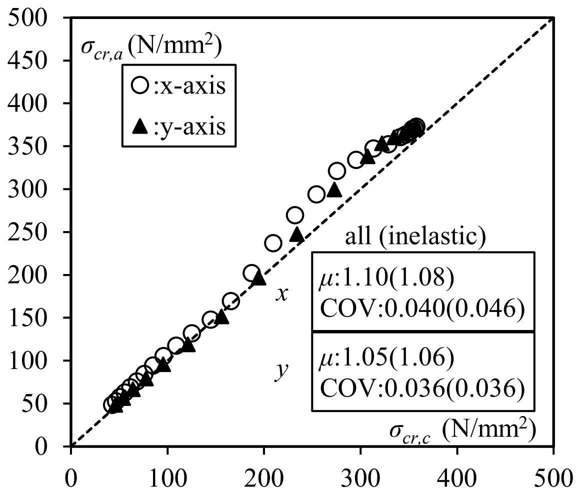
Fig.6 Comparison of analytical and calculated buckling stress (AIJ)



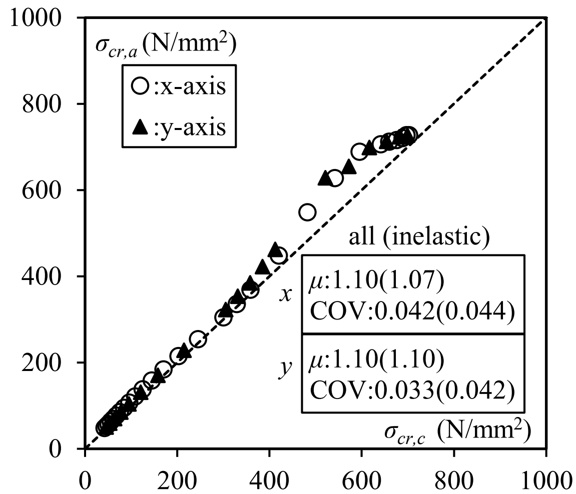
(a) CWF-1



(b) HWF-1



(c) CWF-2



(d) HWF-2

Fig.7 Comparison of analytical and calculated buckling stress (AISC)

CONCLUSION

In this paper, the flexural buckling strength of HWF columns with flange made of ultra-high strength steel and web made of ordinary steel was numerically investigated. From the numerical results, it was found that the current design equations according to AIJ and AISC, calculated based on the equivalent yield stresses of the web and flange evaluate the flexural buckling strength of HWF columns to be equivalent to that of conventional WF-section columns made of fully normal strength steel.

REFERENCES

American Institute of Steel Construction(AISC). (2017): *Specification for Structural Steel Buildings* (ANSI/AISC 360-16)

Architectural Institute of Japan(AIJ). (2019): *AIJ Standard for Allowable Stress Design of Steel Structures*

Asada H, Taga K,Takahashi Y, J. W. Berman, Luo P, Kitaoka T, Tanaka T (2020): "Structural Behavior of Hybrid WF Cross-section Columns Consisting of Ultra-High Strength Steel Flange and Normal Strength Steel Web". 17th World Conference on Earthquake Engineering,

Ban H, Shi G (2018): "A review of research on high-strength steel structures". *Proceedings of the Institution of Civil Engineers-Structures and Buildings*, 171(8), 625-641.

Endo C, Owada S, Taga K, Kitaoka S, Ichinohe Y. (2013): "Study on the Strength Evaluation of Hybrid H-shaped Column". *Proceedings of Constructional Steel*, Vol.21, 345-352

Japan Iron and Steel Federation and Japanese Society of Steel Construction. (2011): "Tokyo Sky Tree -high-strength steel pipe for antenna tower". *Steel Construction Today Tomorrow*, 31, 14-15

Kato A, Taga K, Kitaoka S, Ichinohe Y. (2017): "Aseismatic performance of hybrid steel column consists of high strength steel flange and conventional steel web". *Journal of Structural and Construction Engineering* (Transactions of AIJ), 82(739), 1461–1471.

Kawabata T, Ichinohe Y, Fukuda K, et al. (2011): "Development of 1000N class GMA and SA weld material for buildings". *Quarterly Journal of the Japan Welding Society*, 29(2), 114–124.

Lin X, Okazaki T, Nakashima M (2014): "Bolted beam-to-column connections for built-up columns constructed of H-SA700 steel". *Journal of Constructional Steel Research*, 101, 469-481.

Lin X. (2012): "Development of low-to-mid-rise building structures using weld-free built-up columns made of ultra-high-strength steel", Doctoral Dissertation, Kyoto University.

Sasaki M, Kawabata T, Fukuda K, et al. (2011): "Investigation on application of 1000N class ultra-high-strength steel for the real building". *Proceedings of the Symposium on Welded Structure*, 511-514.

Sato A, Kimura K, Suita K (2009): "Development of weld-free beam-to-column connection of H-SA700A high-strength steel for building structures". *Journal of Structural and Construction Engineering*, 74(646), 2355-2363.

Suita K, Naoki S (2012): "Loading test of H-SA700B column made by undermatched welds and SN490B beam welded moment connection". *Journal of Structural Engineering*, B(58B), 375-383.

Taga K, Tada M, Shirasawa Y, et al. (2012): "Use of ultra-high-strength steel in building frames aiming at major countermeasure for huge earthquakes". The proceeding of 15WCEE, 0254, 1-10.

Tamai H, Yamanishi T, Takamatsu T, Matsuo A (2011): "Experimental study on lateral buckling behavior of weld-free built-up member made of H-SA700A high-strength steel". *Journal of Structural and Construction Engineering*, 76(660), 407-415.

Tsuda K, Kido M, Kawano A (2011): "Strength formula for beam-columns made of H-SA700 high-strength steel". *Journal of Structural and Construction Engineering*, 76(670), 2163-2171.

Xiang P, Ge H, Jia LJ. (2016): "Advancement and applications of Japanese high-performance steel in structural engineering". *Advanced Steel Construction*, 13(2), 245–262.

Yoshida Y, Obinata T, Nishio M, Shiwaku T (2009): "Development of high-strength (780 N/mm 2) steel for building systems". *International Journal of Steel Structures*, 9(4), 285-289.