Ultimate Temperature of Steel Frames Exposed to Fire - Part 3 Influence of Local Buckling on the Ultimate Temperature of Steel Frames -

Takashi Terakawa[1], Junichi Suzuki[1], Hiroyuki Suzuki[2], Takao Wakamatsu[1] and Yoshifumi Ohmiya[1]

[1] *Tokyo University of Science*
[2] *University of Tsukuba*

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1. INTRODUCTION

The resistance against the horizontal force of earthquake was especially added to frames that seismic design was carried out in Japan. It was considered that the resistance influenced fire resistance performance also. And it was clear that frames did not immediately collapse by the stress redistribution of itself after heated columns buckled.[3] However, the influence of local buckling on the stress redistribution was not included in past researches. This had a possibility of reducing the ultimate temperature of frames. In this research, the influence of local buckling on the ultimate temperature was examined.

2. ANALYTICAL CONDITIONS AND BEHAVIOR OF LOCAL BUCKLING

2.1 Stress – strain relation [2]

(1) Stress – strain relation of steel without local buckling was as follows

$$\sigma(\varepsilon)=\sigma_o(\varepsilon)=\begin{cases}\dfrac{E_{lt}\cdot\varepsilon}{1+\left|E_{lt}\cdot\varepsilon/\sigma_{ot}\right|^{\frac{1}{n_t}}}+\dfrac{E_{pt}\cdot\varepsilon}{10\varepsilon+1} & :\varepsilon\le 0.1\\ \dfrac{d\sigma}{d\varepsilon}=0 & :\varepsilon>0.1\end{cases} \tag{1}$$

Here, E_{lt}, σ_{ot}, E_{pt} and n_t were as a function of temperature. And they were determined that they became in good agreement with the experiment results of steel at high temperature.

(2) Stress – strain relation of squared steel stub columns with local buckling was as follows.

$$\sigma(\varepsilon) = \sigma_{LB}(\varepsilon) = f(\varepsilon)\frac{t}{B}\sigma_o(\varepsilon)$$
$$f(\varepsilon) = 2.5/\sqrt{\varepsilon} + 3 \tag{2}$$

The strain local buckling stared was determined by the intersection point of above two equations.

2.2 Analytical conditions

The analytical conditions were as follows.

(1) The elasto-plastic analysis of finite element method was used.
(2) The analysis included the influence of thermal stress.
(3) The temperature of columns and beams increased uniformly.
(4) The heat conduction to the outside of a fire compartment was disregarded.
(5) Fire continued to the collapse of frames.
(6) Steel strength became 0 at 850°C.
(7) Fire did not spread to other compartments.

3. ULTIMATE TEMPERATURE AND THEATRICAL BUCKLING TEMPERATURE

3.1 Ultimate temperature

Frames collapsed by the local buckling of heated columns, or total buckling, and the ultimate temperatures of the frames were governed by both.

Figure 1 showed the relation between the temperature of frames and the deformation angle of beams in the ultimate state. Here, the deformation angle was the value that the contraction of the heating column δ was divided by the length of the beam in the outer span. The local buckling of column occurred at (a) in the figure. When the stress redistribution of the frame shown in *Figure 1.i)* was high, the frame returned to the stable state (b) again after the column buckled locally. Then the ultimate temperature of the frame with local buckling was almost the same temperature that the frame collapsed by total buckling. As shown in *Figure 1.ii)*, even though the frame returned to the stable state (b) again, the ultimate temperature became lower in many cases. It was obvious not to return the stable state again when the deformation angle of beams exceeded 1/50 according to the past research[4]. Therefore, the ultimate temperature of frames was assumed to be the highest temperature ((c) in *Figure 1*) before the deformation angle reached to 1/50.

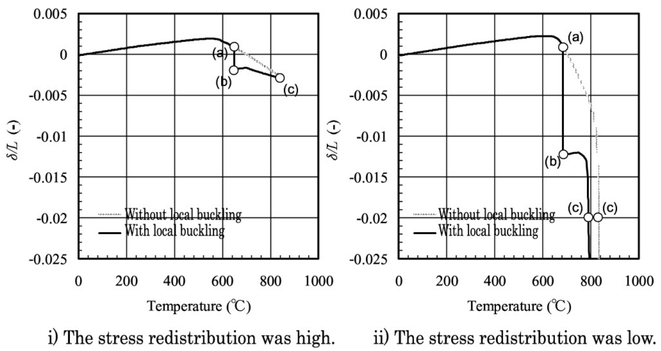


Figure 1 Difference of ultimate temperature

3.2 Theoretical buckling temperature

The theoretical buckling temperatures were calculated by tangent modulus theory for centrally loaded compressed columns.

Heated columns felt into local buckling during with increasing the deformation of buckled column practically, and the local buckling also caused the total buckling. It was obvious that the ultimate temperature of the frames without sway in the collapse of columns related to the buckling temperature of the centrally loaded compressed columns with a length of 0.7 times story height. The effective buckling length of the columns corresponded to the story height of the frames in this paper because of the influence of local buckling caused by increasing of the deformation of total buckling. When the bending resistance of the columns locally buckled was neglected, the resistant force did not decrease. But, increasing the effective buckling length from 0.7 to 1 corresponded to reduction of the buckling strength by superposition of total buckling of local buckling.

4. RESIDUAL STRENGTH OF BUCKLING COLUMN AT HIGH TEMPERATURE

Figure 2 shows a series of residual strength of buckling column at high temperature. The columns had a width thickness ratio of 15 or 33, and a slenderness ratio of 15 or 25. And the temperature of the columns was in the range from 600°C to 800°C. In this figure, the dotted curves showed the residual strength of buckling column without local buckling, and the plots showed the residual strength with local buckling. When the column with *B/t* of 15 contracted at 5%, the strength with local buckling became about 0.5 times strength without it. But, when *B/t* of the column of 33 contracted only at 0.5%, the strength became half of it.

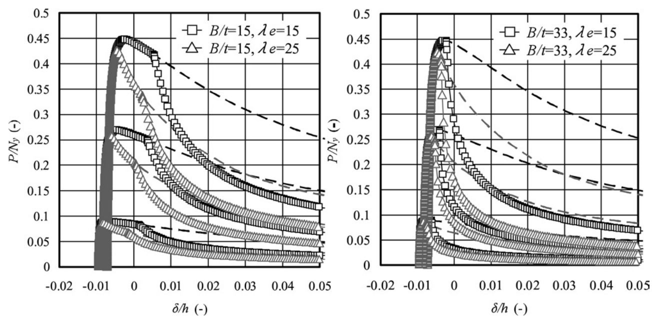


Figure 2 Residual strength of buckling column

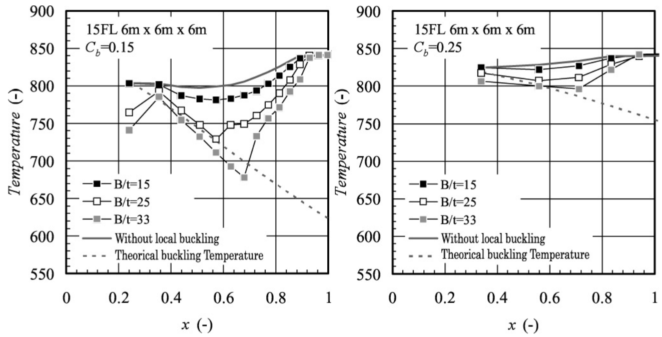
5. FRAME FOR ANALYSIS

The parameters of frames in this analysis were span length, base shear coefficient C_b and *B/t*. The frames with members of a minimum size were designed by floor moment method [3] in order to bear seismic shear force for design. The frames had 3 spans with 15 stories. The story height was 4m. And the span length was a length of 18m (6×6×6m), 26m (10×6×10m) or 34m (14×6×14m). *B/t* was 15, 25 or 33. C_b was 0.15 or 0.25. Moreover, load on beams was 47.0 kN/m. it corresponded to floor load of 7.84 kN/m^2.

6. ANALYSIS RESULTS AND CONSIDERATION

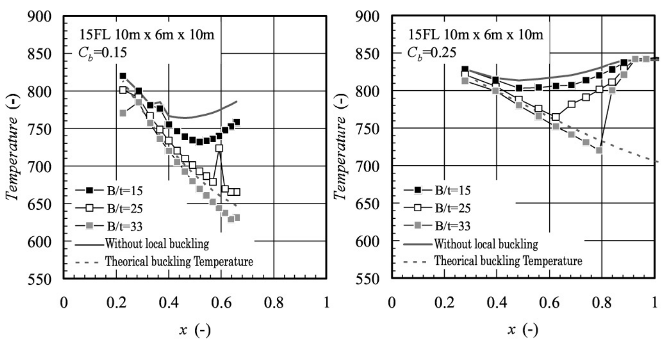
Figure 3 to *5* show the ultimate temperatures of the frames with the difference in *B/t*, C_b and the span length. The solid curves were the ultimate temperatures of the frames without local buckling in this figure. The plots in black, white and gray were the ultimate temperatures of the frames with B/t of 15, 25 and 33, respectively. The dotted curves were the theoretical buckling temperature. Analysis results with x of over 1 were excepted because the frames did not fall into overall collapse and the ultimate temperature reached to 850°C. The ultimate temperatures of frames with large *B/t* decreased even though the frame had high C_b. Because the residual strength of columns with large *B/t* immediately decrease as shown in *Figure 2*.

Especially, when x was almost in the range from 0.4 to 0.8, the ultimate temperatures decreased remarkably. But, when x was smaller than 0.4 or exceed 1, *B/t* did not influence the ultimate temperature. It indicated that the influence of local buckling became remarkable when the frames felt into collapse after the stress redistribution was fully used. Comparing with the theoretical buckling temperature shown in Section 3, the ultimate temperatures has exceeded the theoretical buckling temperatures except for a part of frames with *B/t*=33, and if frames used members with rank A, the lower limit of the ultimate temperature was estimated by the theoretical buckling temperature. The ultimate temperature in top floor fire was less than the theoretical buckling temperature because the deformed beam pulled the exterior column into the inside. The influence of local buckling caused the decrease of the ultimate temperature, but the stress redistribution increased the ultimate temperature.



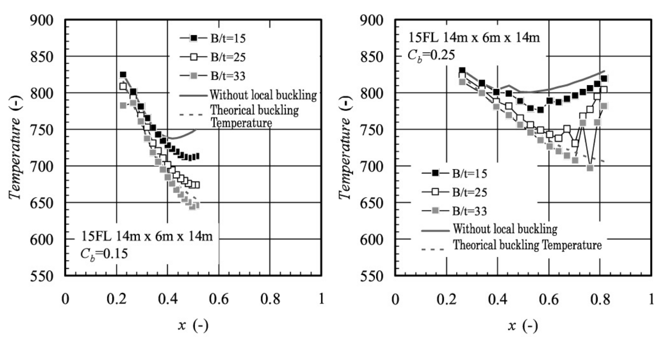
a) C_b=0.15 b) C_b=0.25

Figure 3 Relation between stress redistribution rate x and ultimate temperature (18m(6×6×6m))



a) C_b=0.15 b) C_b=0.25

Figure 4 Relation between stress redistribution rate x and ultimate temperature (26m(10×6×10m))



a) C_b=0.15 b) C_b=0.25

Figure 5 Relation between stress redistribution rate x and ultimate temperature (34m(14×6×14m))

REFERENCES

1. Architectural Institute of Japan, "Recommendation for Fire Resistant Design of Steel Structure", 1999 (in Japanese)
2. Kuma T., Suzuki H. and Ide Y., "Ultimate Temperature of Steel Frames Subjected to Fire, Part22 Analysis of Local Buckling Behavior of Beam-Column", Summaries of technical papers of Annual Meeting Architectural Institute of Japan. A-2, Fire safety, off-shore engineering and architecture, information systems technology, pp.69-70, 1998 (in Japanese)
3. Suzuki J., Abe S., Suzuki H., Ohmiya Y. and Wakamatsu T., "Ultimate Temperature and Structural Redundancy of Steel Frames Exposed to Fire -Effect of Seismic Design on Fire Resistance-", Journal of structural and construction engineering. Transactions of AIJ, No.608, pp.157-164, 2006 (in Japanese)
4. Suzuki H., Terakawa T., Suzuki J., Ohmiya Y., and Wakamatsu T., "Ultimate Temperature of Steel Frames Exposed to Fire -Part 2 Stress Redistribution and Limit of Deformation of Steel Frames-", Summaries of technical papers of Annual Meeting Architectural Institute of Japan. A-2, Fire safety, off-shore engineering and architecture, information systems technology, pp.145-146, 2007 (in Japanese)