Ultimate Temperature of Steel Columns subject to Thermal Elongations of Adjacent Beams at Fire

by

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ABSTRACT

The present paper focuses on structural fire resistance of heated steel columns, in particular the columns subjected to thermal elongations of adjacent beams at fires. Behavior of the steel column at the fire is minutely investigated by making full use of a refined finite element analysis taken a 3-D non-linear thin plate element into account. From results obtained by numerically parametric calculations, it is clarified that the above thermal elongation is not a sensitive factor affected the fire resistance of the steel column and its ultimate temperature can be estimated by verification methods which has been proposed in the Recommendation for Fire Resistant Design of Steel Structures (AIJ, 1999).

KEYWORDS: FEM, Local buckling, Overall buckling, Steel column, Thermal elongation of beam, Ultimate temperature

1. INTRODUCTION

Recent studies on the fire resistance of steel structures have focused on the stability of steel members and steel structures during fires [1-5]. To investigate the structural stability of structures exposed to fire, the fire load-bearing capacities of key members (beams, columns and connections) and other factors involved in their stability should be investigated since their collapse may lead to the collapse of the entire structure. The overall structural stability during fires of the combined members needs to be understood. Since the fire stability of members is closely related to the fire stability of the frames, the fire stability of both should be known to determine the fire resistance of a structure.

Overall and local buckling of steel columns are one of the key causes that destabilize a structure. Especially, the fire resistance of columns positioned within frames is affected by elongation of adjacent beams. When beams are heated during a fire, the beams expand lengthwise and push adjacent heated columns. The longer the beam, the further it elongates, which may cause local buckling of adjacent columns at their upper and lower ends. Structures with long beam spans have increased in recent years and could suffer excessive elongation during fires. In most structures, the collapse of a fire compartment room is controlled by applying fire proofing materials to steel members to prevent excessive elongation and using fire resistant columns. Methods for controlling the heat input into beams have been found to be effective for preventing elongation, but the necessary structural fire resistance of columns is not understood and this question has not yet been clearly resolved.

In the present study, a numerical analysis by the finite element method was conducted to investigate factors involved in the destabilization of steel columns at high fire temperatures, such as the possibility of local buckling when pushed by elongating beams, the behaviors of steel columns after local buckling, and the possibility of local buckling inducing failure of the columns, and to determine the temperature at which the columns collapse [1]. The results of the numerical analysis were also used to investigate the validity and feasibility of existing methods for assessing the fire resistance of steel columns.

In Japan, various attempts have been made to ensure the fire safety of steel structures, such as the method for assessing the collapse temperature at which steel columns suffer local buckling proposed in “ Recommendation for Fire Resistant Design of Steel Structures ”[6]. The validity of the method has been investigated by

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conducting non-linear finite element analysis [7] using two-dimensional beam elements, in which several numerical assumptions are made, such as the deteriorating stress-strain relationship after local buckling is given as a known value and the beam element models are restricted to two-dimensional behaviors on a plane. Actual steel columns have cross sections in which steel plates are combined, so local buckling involves the three-dimensional behavior of collapse of the sectional form. To investigate the validity of the method for determining the collapse temperature, a precise numerical analysis is needed. This paper describes a non-linear finite element analysis using three-dimensional thin plate elements (shell element) to investigate the precise behaviors of steel columns during local buckling at high temperatures. The validity of the analysis was examined by comparing the results with those of high-temperature experiments of short steel columns in the past.

2. COMPARISON OF NUMERICAL ANALYSIS RESULTS FOR SHORT STEEL COLUMNS AND EXPERIMENTAL RESULTS IN THE PAST

Hirashima et al. conducted compressive tests of heated stub columns at a constant temperature and under increasing load [8]. In their experiment, weld assembly square hollow columns were used. A cross section of the specimen is shown in Figure 1, and its specifications are listed in Table 1. The plate width of the specimen was B, the plate thickness was t, and the total length was H. Specimens of width to thickness ratios (B/t) of 25 and 30 were used. The length to width ratio (H/B) was 3 in all specimens. In the experiment by Hirashima et al., splice plates were welded on the corners to prevent the weld joints from breaking during the experiment (Figure 1). The thickness of the splice plates (t_s) was 9 mm, which was thicker than the plates constituting the specimens (t). Therefore, the experimental results should have been affected by the splice plates. The models of the rectangular specimens used for numerical analysis are shown in Figure 2(a) and 2(b). The finite element models were constructed using the three-dimensional thin plate elements

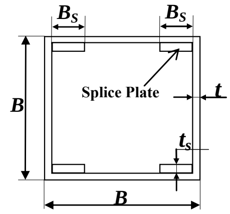
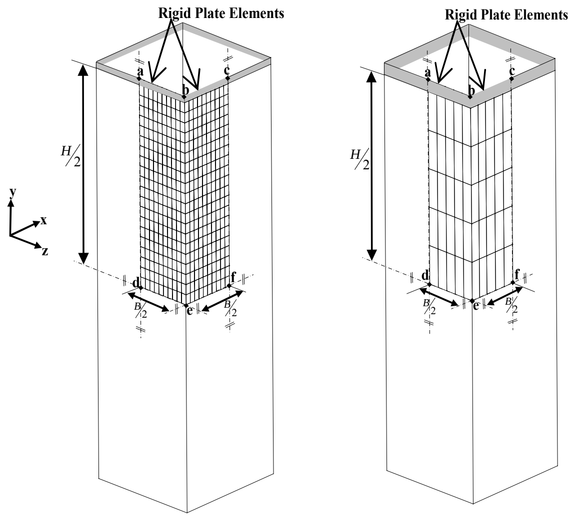


Fig-1 Section of Specimen

Table 1 Specifications of Specimen

	B/t = 25	B/t = 30
Plate Width B	150	135
Plate Thickness t	6	4.5
Height of Specimen H	450	405
S. Plate Width B_s	25	25
S. Plate Thickness t_s	9	9

(Unit : mm)



(a) Model A **(b) Model B**

Fig-2 Analytical Models of Stub Columns

proposed in Reference 9). Details of the models are described in the reference; an overview is given below:

1) The plate elements were rectangular. There were eight nodes per element: four on the corners and four in the middle of each side. The nodes on the corners had two degrees of freedom along the parallel direction, two degrees of freedom along the out-of-plane rotational direction, and two degrees of freedom along the in-plane rotational direction, making a total of seven degrees of freedom. The nodes in the middle of the sides had one degrees of freedom along the parallel direction. Each element had a total nodal degrees of freedom of 32.
2) The strain in the elements was determined according to Green's strain tensile [10] to cope with geometrical non-linear problems. The multi-axial elasto-plastic constitutive law followed the Prandtl-Reuss law [11] and was used to derive the increment stress-strain relationship under plane stress.

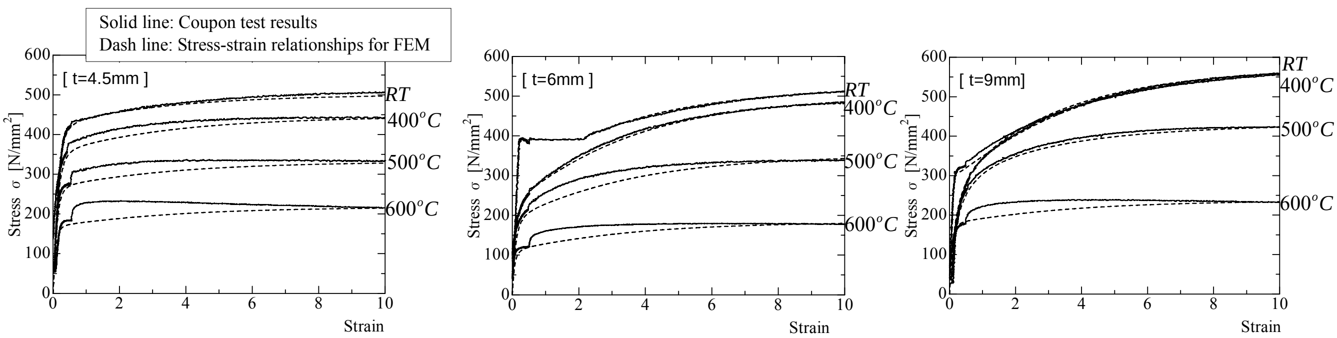
3) The equation of equilibrium of force for the plate elements was determined using the principle of virtual work. The discretization into finite elements, and derivation of incremental control equation (incremental relationships between the node force and node displacement) were determined.

Assuming the specimens were symmetrical right and left and up and down, partial analytical models A and B, each consisting of two steel plates, shown in Figure 2(a) and 2(b) were analyzed. The number of elements differed between Models A and B. Model A was divided into 400 elements of equal size, and Model B was divided into 50 elements of equal size. The other analytical conditions were all the same. Models A and B were used for the following reasons.

Generally, local buckling deformation in the columns tends to concentrate at points of buckling. At the points of local buckling, the plate deforms toward the out-of-plane, showing very complicated stress conditions. Thus, at least a certain number of elements are needed to precisely analyze the behavior of plates during local buckling. On the other hand, the calculation load jumps when the number of three-dimensional plate elements is increased. For example, parametric numerical analysis of the behaviors of steel columns during fire, which is described in a following paragraph, requires an enormous amount of calculation and long calculation time. The three-dimensional plate elements used in this numerical analysis can precisely analyze the non-linear behaviors of plates using the least number of elements because each element has a nodal degrees of freedom of 32, enabling the displacement function inside the element to be set more precisely than for plate elements that have only four nodes [12)]. This was expected to reduce calculation loads. To examine the advantage, Model B shown in Figure 2(b) was also used as an analytical model.

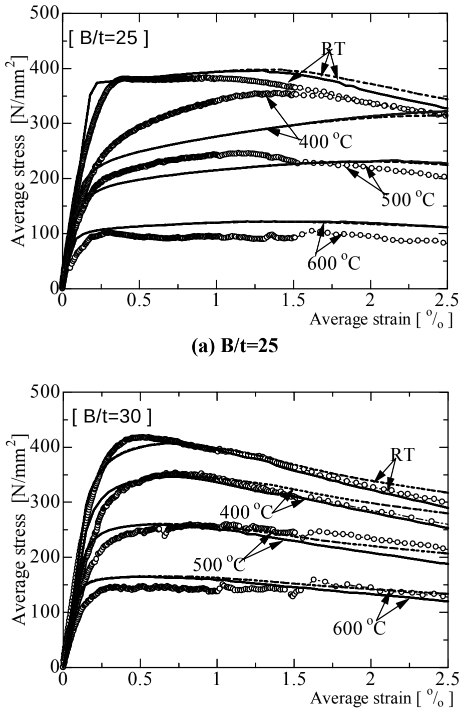
The relationships between stress σ and strain ε at high temperatures used in the analysis are shown in Figure 3. The stub column specimens were made of JIS SM490 steel, and the test results are shown with solid lines in the figure. The dashed lines in the figure denote the σ-ε relationships used in the numerical analysis, which were derived by approximating the test results (solid lines) to the σ-ε relationships of steels stated in the Recommendation for Fire Resistant Design. Young's modulus of the steel at high temperatures was the values stated in Reference 6). Figure 3 shows that the stress jumped at a strain of 0.5% at temperatures above 500°C. This was because the strain speed changed from 0.3%/min to 10%/min at 0.5% strain. This sudden rise in stress was not incorporated in the σ-ε relationships (dashed lines) used for the numerical analysis. At strains of over 0.5%, the forms of the σ-ε relationships (dashed lines) were determined so as to agree with the experimental stress values at 10% strain. This method was based on the same concept used by Hirashima et al. for determining the σ-ε relationships. Hirashima et al. reported no specific values for the loading speed (strain velocity) during their short column experiment at high temperatures but only that the loading was continued for about 100 minutes. The loading speed estimated from the duration of loading was slower than that used for the coupon tests at high temperatures (0.3%/min).

The sections where the splice plates were welded were modeled as plate elements that had



(a) Thickness t=4.5 mm (b) Thickness t=6.0mm (c) Thickness t=9.0mm

Fig-3 Stress and strain relationships



(a) B/t=25

(b) B/t=30

Fig-4 Comparison between experimental and analytical results

a thickness of the sum of the plate thickness t of the specimen and the plate thickness t_s of the splice plate. Since both ends of the specimen were connected to an accelerator via steel blocks, rigid plate elements were added to the upper ends of Models A and B, and the rotational freedom along the upper sides ab and bc was restricted. The initial imperfections were given by applying minute uniform load on the sides de and ef at the center of the specimen in Figure 2 toward the out-of-plane direction of the plates. The uniform load was adjusted so as to produce initial out-of-plane deflection of -B/1,000 and B/1,000 at points d and f in the middle of the specimen, respectively, in all analytical cases. The load imperfections were given in this analysis because it was easier than giving configuration imperfections. The stub column specimens tested were suspected of having produced local temperature distribution and strain restriction, and the specimens before load application at the high temperature were likely to have received any minute load imperfections by heat and other stresses as well as configuration imperfections. The displacement at point b on the upper end along the member axis (displacement along the y direction in Figure 2) was controlled, and uniform compressive load was applied on the specimen via rigid plate elements. Analysis was conducted for specimen temperatures ranging from ambient temperature to 600°C.

The experimental results and the results of the numerical analysis are shown in Figure 4 for stub columns of square hollow section. The X axis of the figure shows the average strain of the specimens, and the Y axis shows the average stress. The former is the value determined by dividing the total length H of the specimen by compressive deformation δv, and the latter is the value determined by dividing the compressive force P applied on the specimen by the total cross sectional area A of the specimen including the area of the splice plates. The open circles in the figure denote the experimental results, and the solid and dashed lines are the analytical results of Models A and B, respectively. Figure 4(a) and 4(b) show that the experimental and analytical results agreed with each other quite well in all cases although not perfectly. Especially at B/t = 30 (Figure 4(b)), the ultimate strength of the specimens and their behaviors after the peak agreed very well. When the analytical values of Models A and B (solid and dashed lines) in Figure 4(a) and 4(b) are compared, the results of Model B (dashed line) were slightly higher than those of Model A (solid line). Model B, which had smaller elements than Model A, showed coarser deformation at the points of local buckling than in Model A. However, as shown in Figure 4(a) and 4(b), the analytical data of Model B agreed well with the experimental results.

3. ESTIMATING THE ULTIMATE TEMPERATURE OF STEEL COLUMNS UNDER THE EFFECTS OF ELONGATING BEAMS

The following three methods were used to estimate the temperature at which steel columns collapse. The methods have been proposed for estimating ultimate temperatures. The relationships with this study and the importance of the methods are described below.

3.1 THEORETICAL BUCKLING TEMPERATURE T_B OF COLUMNS BY THE

Fig-5 Overall buckling of a column with pin supports at the upper and lower ends

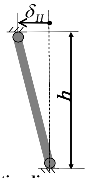


Fig-7 Schematic diagram of a column with local buckling at both upper and lower ends

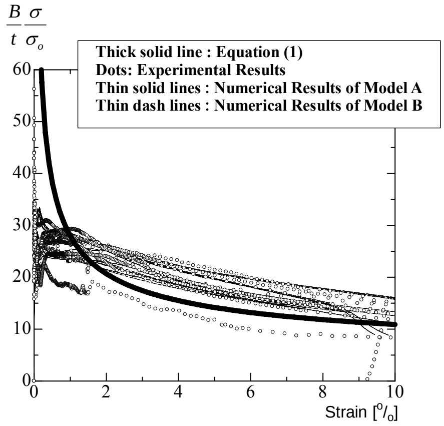


Fig-6 Residual strength of stub columns after local buckling

The Recommendation for Fire Resistant Design of Steel Structures by AIJ proposes equations for estimating the ultimate temperatures at which a steel column compressively collapses by local buckling (hereinafter referred to as the "AIJ equations"). The recommended equations estimate the residual strength σ of the steel column after local buckling by:

$$\frac{\sigma}{\sigma_o} = \left(\sqrt{\frac{6.35}{\varepsilon}} + 3\right)\frac{t}{B} \qquad (1)$$

TANGENT MODULUS THEORY

The temperature at which the steel column buckles during fire can be estimated from theoretical buckling temperature T_B using the tangent modulus theory [5,6]. To estimate T_B, the effective buckling length of the column at high temperatures should be determined. The Euro Code [13] recommends to use 0.5 as the effective buckling length coefficient γ of a steel column at high temperatures for columns when there are members of ordinary temperatures adjacent to both the upper and lower ends and to use 0.7 for columns on the top story. On the other hand, the Recommendation for Fire Resistant Design of Steel Structures by AIJ proposes to use 1.0 regardless of the boundary conditions of the column at both ends since elongation of adjacent beams may cause local buckling of the column at the upper and lower ends and loosen the rotational restriction of the column [6]. Assuming that the column loses bending strength at the points of local buckling but retains the capacity to sustain axial load, the said points can be assumed to be pin supports (Figure 5). In this case, the buckling length coefficient is 1.0. In this paper, the coefficient was decided to be $\gamma = 1.0$ by considering the possibility of local buckling.

3.2 ULTIMATE LOCAL BUCKLING TEMPERATURE T_{LB1} OF STEEL COLUMNS IN THE RECOMMENDATION FOR FIRE RESISTANT DESIGN OF STEEL STRUCTURES BY AIJ

where, σ_o is the stress determined from the relationship between σ and ε in the tensile region. Equation (1) was proposed by Suzuki and Segawa et al. for estimating the residual strength of rectangular steel after local buckling [14]. They performed a number of compression tests of stub columns at high temperatures, extracted and analyzed the dynamic significance of behaviors during local buckling, and found that the equation (1) estimated by a lot of experimental results did not much depend on test temperature, the ratio of width to thickness, and material properties. The residual strengths of the steel columns after local buckling, which were calculated using Equation (1), are shown in Figure 6 together with the experimental and analytical results of the stub columns described in Section 2. The horizontal axis ε of both experimental and analytical results is differently evaluated before and after the ultimate strength. Before the ultimate strength, strain was assumed to be uniformly distributed throughout the specimens ($\Delta\varepsilon = \Delta\delta_V / H$, Δ denotes differences). After the ultimate strength, strain was assumed to concentrate at points of local buckling, which spread over the plate width B ($\Delta\varepsilon = \Delta\delta_V / B$) [8]. Figure 6 shows that all the experimental and analytical results converged into almost a single curve and Equation (1) for calculating residual strength after local buckling gives a curve that lies along the lower limit of the experimental and analytical values.

The AIJ equations assume that a strain ε_0 of 0.01 is generated along the edge on the tensile side of a column section where local buckling occurs and a strain ε_1 of 0.04 is generated along the edge on the compression side. The strain distribution in a cross section is given by a line. The strain on the compression side of the section is increased by elongation of adjacent beams. The recommended equations assume a strain ε_1 of 0.04 for the edge on the compression side by considering the effects of the beams. Using Equation (1), the stresses in the section are integrated, and the temperature of the members at which the resultant value is equal to the axial compressive stress of the column, which is the external force, is determined. The temperature is the ultimate local buckling temperature of the column [6] and is hereinafter referred to as T_{LB1}.

3.3 ULTIMATE LOCAL BUCKLING TEMPERATURE T_{LB2} OF COLUMN UNDER EFFECTS OF ELONGATING ADJACENT BEAMS

Joint translation angles are formed on columns when adjacent beams elongate. Joint translation angles are larger in longer spans and at higher member temperatures. Excessive joint translation angles induce early local buckling and accelerate the deterioration of the column after buckling. The theoretical buckling temperature T_B and the ultimate local buckling temperature T_{LB1} of the AIJ equation are ultimate temperatures determined by the date on the only column and are independent of joint translation angles. Thus, when the joint translation angle is excessively large, the column may collapse at a temperature lower than the ultimate temperatures.

Suzuki and Nakayama et al. have proposed an equation for calculating the ultimate local buckling temperature of a column under the effects of elongating beams (hereinafter referred to as the "original Suzuki Equation") [7]. A schematic diagram of local buckling of a column by the elongation δ_H of an adjacent beam is shown in Figure 7. Local buckling was assumed to occur at the upper and lower ends of the column and to rotate like a hinge. The original Suzuki equation assumed local buckling only at the upper end of a column, but large δ_H may also cause local buckling at the lower end. Thus, a column was assumed to have two local buckling hinges. The equation for calculating the ultimate temperature by assuming two hinges is hereinafter referred to as the "modified Suzuki equation". The length of the hinge that becomes plastic was assumed to be equal to the buckling wavelength at the buckling points, and the wavelength was assumed to be equal to the sectional plate width B. The resultant difference between the strain ε_1 on the compression side and the strain ε_0 on the tensile side becomes equal to the hinge rotation θ:

$$e = \theta = \frac{\delta_H}{h} = \alpha T \frac{L}{h} \qquad (2)^{7)},$$

where, L is the total span length of the heated beam, α is the elongation coefficient of steel (= 12×10^{-6} 1/°C), h is the height of the column in the story, and T is the material temperature. Thus, the modified Suzuki equation for determining the strain ε'_1 on the compression side is:

$$\varepsilon_1' = \varepsilon_o + e = \varepsilon_o + \alpha T \frac{L}{h} \qquad (3).$$

The strain ε_0 on the tensile side is given as 0.01 as in the recommended equation. The process of determining the ultimate temperature is the same as that used in the recommended equation. The ultimate local buckling temperature determined by the modified Suzuki Equation is referred to as T_{LB2}.

4. NUMERICAL ANALYSIS OF A COLUMN AFFECTED BY AN ELONGATING BEAM

4.1 MODEL USED FOR NUMERICAL ANALYSIS

The model used for numerically analyzing a column under the effect of an elongating beam is shown in Figure 8. The model is a part of the frame model formed by removing the exterior columns and beams of several spans from the model of the frame in the story where there is a fire (Figure 9). A uniform compressive force P was applied on the column, and the member temperatures T of the column and beam were raised uniformly. The boundary conditions at the lower end of the column were fixed. The heated beam in Figure 8 was assumed to be rigid at ordinary and high temperatures and to expand horizontally as the temperature rose for a length of δ_H. The effective buckling length coefficient γ was 0.5 (theoretical value for a column with

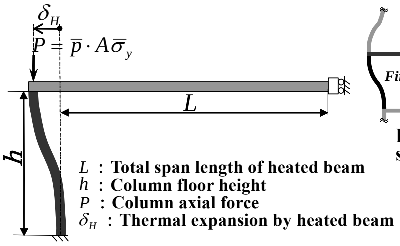


Fig-8 Numerical analysis model of a column affected by an elongating beam

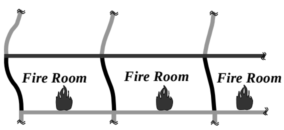


Fig-9 Fire expanding to several spans

Table-2 Parameters of Numerical analysis

$\overline{p}$	0.1、0.3、0.5
B/t	20 (□−500×500×25) 25 (□−500×500×20) 33 (□−500×500×15)
L/h	1～15

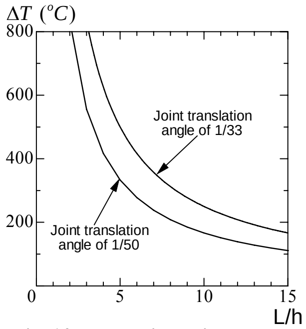


Fig-10 Relationship between L/h and ΔT

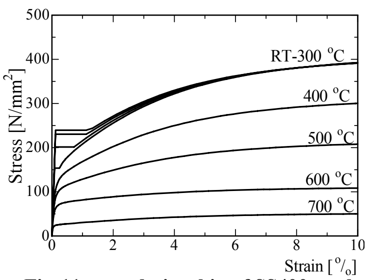


Fig-11 σ–ε relationship of SS400 steel

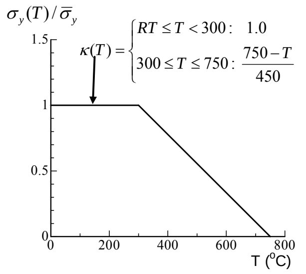


Fig-12 Reduction factor of SS400 Steel

both ends fixed) when no local buckling occurred. During an actual fire in a frame, the upper and lower ends of buckling columns on the story of the fire are believed to be strongly restricted by members on the other stories, which are at ordinary temperatures (Figure 9).

Parameters used for the numerical analysis are shown in Table 3. Three axial force ratios $\overline{p}$, three column width to thickness ratios B/t, and beam to column length ratios L/h ranging from 1 to 15 were used as the analytical parameters. The column height h was fixed to 4,500 mm, and L/h was varied by changing the span length of the heated beam. The width to thickness ratio was varied (B/t = 20, 25, and 33) by fixing the plate width B of the square cross section and changing the plate thickness t. B/t = 33 is the limit width to thickness ratio of square hollow steel columns (JIS SS400) of Class FA at ordinary temperatures [15]. Since the floor column height h and the sectional plate width B were set uniform and only plate thickness t was changed, the height to width ratio λ of the column was almost uniform in all analytical cases (λ = 0.25). Therefore, the three columns were similarly resistant against overall buckling. The analytical case with a large L/h value assumed a fire in a space with a long beam span and a fire spreading to a large space on a story by burning partition walls (Figure 9). The relationships between L/h and temperature rises ΔT are shown in Figure 10. The figure shows that, for example, for L/h = 15, a temperature rise of about 110°C causes a joint translation angle $\Delta\delta_H/h$ of 1/50.

The steel was assumed to be JIS SS400. Its σ–ε relationships at high temperatures are shown in Figure 11. These are the σ–ε relationships proposed by the Recommendation for Fire Resistant Design of Steel Structures. At each temperature, the 1% stress σ_y corresponds to $\kappa(T)\overline{\sigma}_y$ (Figure 12). $\overline{\sigma}_y$ is the standard yield strength of steel at ordinary temperature. The analysis considered no effects from the creep of steel at high temperatures to examine the validity and feasibility of the method for estimating the ultimate temperature described in the previous section. In the evaluation method, the effects of creep should be investigated separately or the effective yield strength of steel at high temperature should be reduced when the effect is considered (Figure 12). Thus, the effects of creep of steel columns at high temperature are not discussed in this paper but will be separately reported. The aforementioned three-dimensional plate elements were used as the finite element model, which was a square hollow section consisting of four steel plates. Each plate was divided into elements using the methods described in Section 2 for dividing Model B of a stub column specimen into elements. In the column model shown in Figure 8, the plates were divided into 10 and 30 elements of equal sizes along the sectional and axial directions, respectively. The total number of elements was 1,200. Actual loading models should have been different between the beam-column model shown in Figure 8 and the stub column model shown in Figure 2, the former of which should have been more complicated that the latter. Although the analysis was a pure compression analysis, the minute elements of the plates after local buckling were under very complicated stress conditions and showed three-dimensional non-linear behaviors of in-plane and out-of-plane deformation. The numerical analysis of Model B of the stub column precisely reproduced the complicated non-linear behaviors of plates. Therefore, the element division method used for Model B was used for the column model shown in Figure 8. The numerical analysis case of B/t = 25, $\overline{p}$ = 0.3, and L/h, which is discussed in the following section, was analyzed using the element division method used for Model A in Figure 2. The analysis results in an ultimate temperature difference of only 2°C from the result of an analysis conducted using the element division method of Model B.

4.2 RESULTS OF THE NUMERICAL ANALYSIS OF A COLUMN AFFECTED BY AN ELONGATING BEAM

The results of the numerical analysis of all parametric study cases are shown in Figures 13 to 15. Figures 13, 14, and 15 show the results of B/t = 20, 25, and 33, respectively. The X axis of the figures is the beam to column length ratio L/h, and the Y axis is material temperature T. The dark dots in the figures are the collapse temperatures of the column given by numerical analysis of three-dimensional plate elements (hereinafter referred to as the “numerical ultimate temperature”). For analytical cases in which local buckling occurred in the process of heating, the member temperatures at which the local buckling occurred are shown with double circles. Local buckling temperatures are the member temperatures when out-of-plane buckling waveforms were produced in a plate at the upper or lower end of the column. The numerical ultimate temperatures determined by using a two-dimensional non-linear beam element model as a finite element model are also shown with open triangles in Figures 13 to 15. The same partial frame model (Figure 8) was used for the numerical analysis using the beam element model to the calculated ultimate temperature. The same analytical parameters (Table 3) and σ–ε relationships (Figure 11) were also used. The beam element model is not for analyzing the local buckling behaviors and considers no drop in strength by local buckling. Thus, the collapse mode was overall buckling of the columns for all cases. Figures 13 to 15 also show theoretical ultimate temperatures of various kinds. The ultimate local buckling temperature T_{LB2} determined by the modified Suzuki equation is shown as a solid line. The ultimate local buckling temperature T_{LB1} determined by the AIJ equation is shown as a dashed line. The theoretical buckling temperature T_B for an effective buckling length coefficient γ of 1.0 is shown as a dotted line. The trends shown by the series of parametric numerical analyses and comparisons with the theoretical ultimate temperatures are discussed in the following section.

[1] First, an analytical case of a column affected by an elongating beam is described. The analytical results of B/t = 25 and $\overline{p}$ = 0.3 are shown in Figure 16(a) and 16(b). Figure 16(a) is for L/h=1, and Figure 16(b) is for L/h = 15, both of which show the deformation when the columns collapsed. The elongation δ_H of the beam at the upper end of the column is produced along the +y direction in the figure. The temperature history of vertical displacement (displacement along the x direction in Figure 16) of the column head calculated from the two analyses is shown in Figure 17. Figure 17 shows upward extension of the column of L/h = 1 by itself thermal expansions. On the other hand, the column of L/h = 15 shows a drop in vertical displacement at the column head as the temperature rose since the settlement of the head by inclination was larger than the extension of the column. On the column of L/h = 15, local buckling occurred at the plates on the compression side at the upper and lower ends of

[Legends for Figures 13 to 15]

● : Numerical ultimate temperatures(using three-dimensional plate elements) — Solid lines: T_{LB2}

◎: Numerical local buckling temperatures(using three-dimensional plate elements) — Dash lines: T_{LB1}

Δ : Numerical ultimate temperatures(using two-dimensional beam elements) — Dotted line: T_B

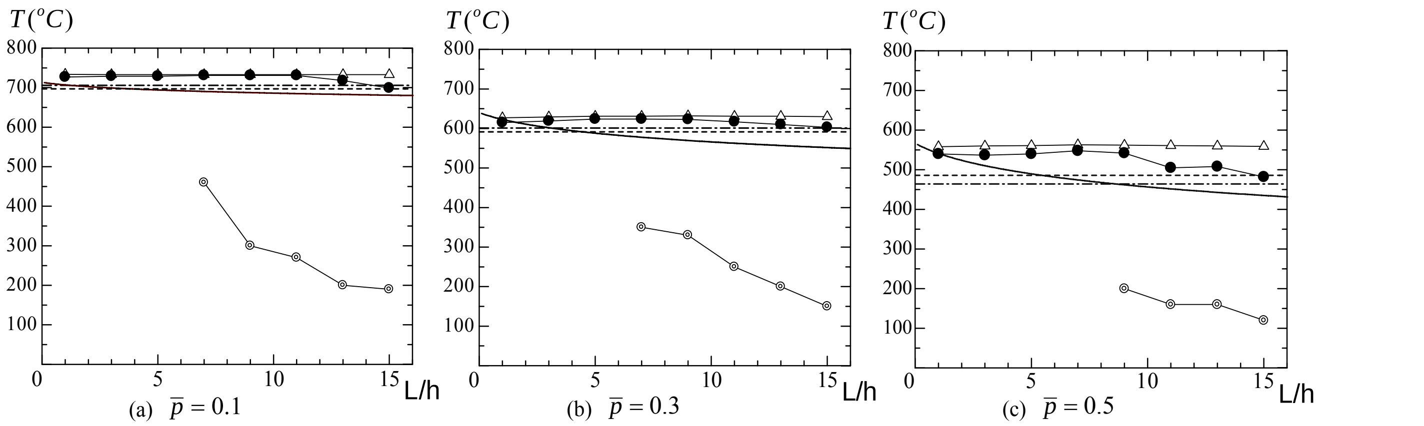


Fig-13 Numerical Results [B/t=20]

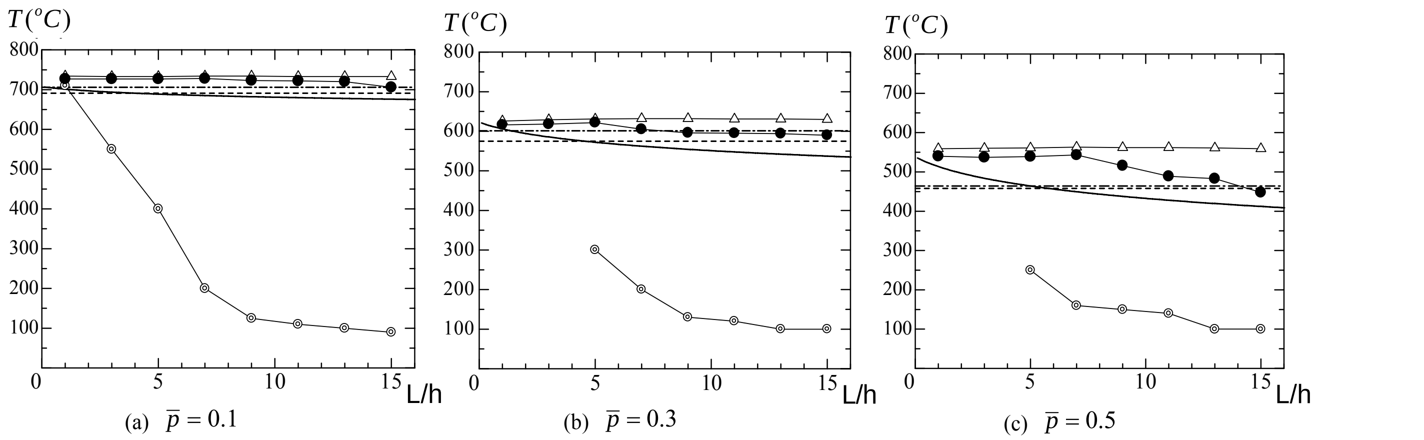


Fig-14 Numerical Results [B/t=25]

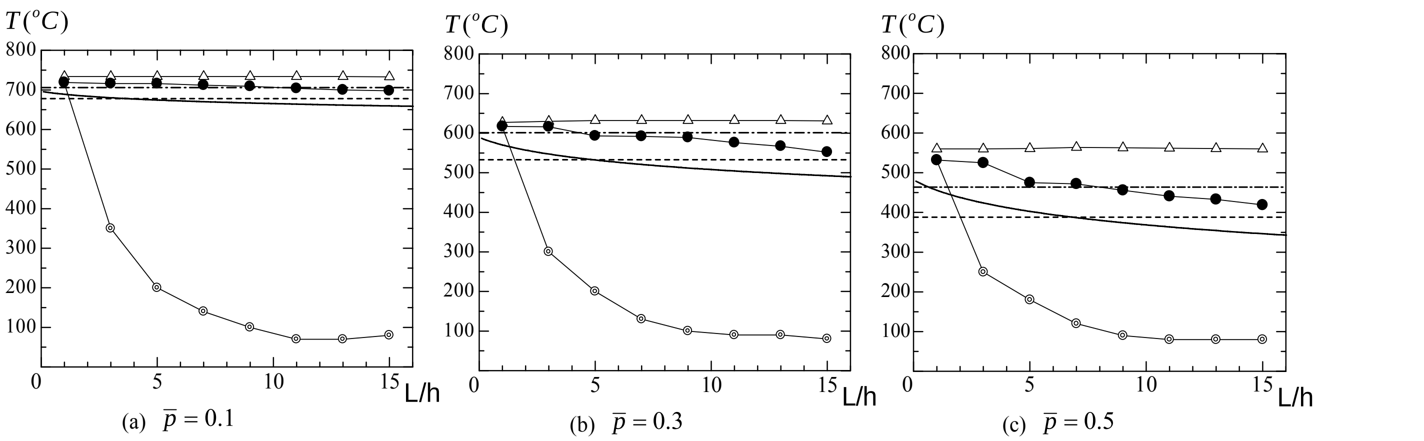
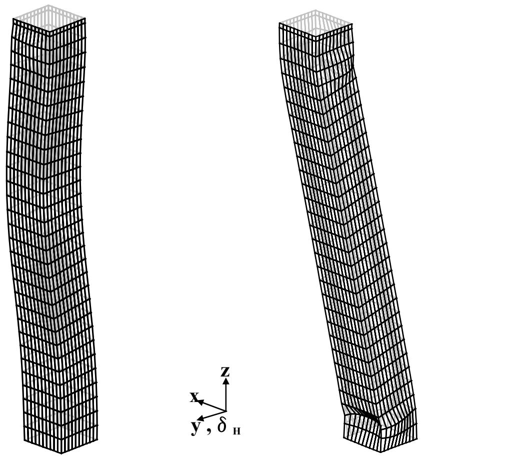


Fig-15 Numerical Results [B/t=33]

the column at a member temperature of about 100°C. The column could sustain the fixed axial force P even after the local buckling, and the member temperature rose further. In both analyses, immediately before the collapse of the columns, the column head suddenly sank, and



(a) $L/h = 1$
(Overall buckling, ultimate temperature: 616°C)
Magnification of deformation: 5

(b) $L/h = 15$
(local buckling, ultimate temperature: 616°C)
Magnification of deformation: 2

Fig-16 Deformation of a column at collapse (analytical result for $B/t = 25,\ \bar{p} = 0.3$)

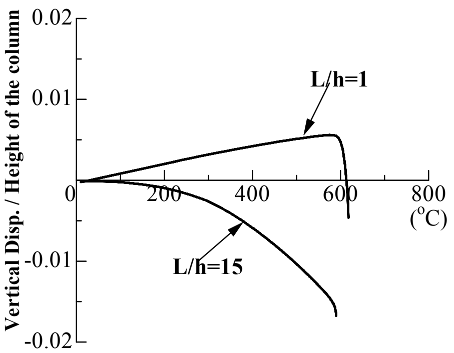


Fig-17 Displacement of column head along the material axis ($B/t = 25,\ \bar{p} = 0.3$)

the columns became unable to sustain the axial force and collapsed. This member temperature was the ultimate temperature of the column, and is the limit member temperature at which the column can keep the equilibrium of forces. The ultimate temperature of the column of L/h = 1 was 616°C, and that of the column of L/h = 15 was 590°C. In Figure 17, the ultimate temperature is around the member temperature at which the column head showed a sudden vertical displacement. The mode of collapse was overall buckling in the column of L/h = 1 (Figure 16(a)) and was compressive collapse at the points of local buckling in the column of L/h = 15 (Figure 16(b)). As Figure 17 shows, the entire column of L/h = 1 suffered buckling deformation and showed jumps in the settlement of the head near the ultimate temperature. On the other hand, in the column of L/h = 15, local buckling waveforms concentrated at the upper and lower ends of the column, and the settlement of the head at around the ultimate temperature was smaller than that of L/h = 1. Figure 16 shows a bow-shaped buckling deformation of the column of L/h = 1 and local buckling deformation of the column of L/h = 15. The parametric study of the analytical cases showed that the mode of collapse of columns in which local buckling occurred during the process of heating was compressive collapse by local buckling and that of columns in which no local buckling occurred was overall buckling.

[2] Figures 13 to 15 showed that local buckling occurred during the process of heating in many cases analyzed (○). The temperature at which local buckling occurred was lower in cases of longer beam elongation, i.e. with larger beam to column ratio L/h. Especially, when both L/h and the width to thickness ratio B/t were large, local buckling occurred at temperatures below 100°C (Figure 15). However, the local buckling did not cause the column to collapse immediately. The steel column kept stable residual strength even after it suffered local buckling (Figure 6) up to high temperatures. As the temperature rose, the joint translation angle gradually increased and accelerated the strain at the points of local buckling on the compression side, and the column ultimately collapsed by compressive fracture at the points of local buckling. The numerical analysis showed that the ultimate temperatures of columns (●) gradually dropped as L/h and B/t increased. The relationship between the column axial force ratio $\bar{p}$ and calculated ultimate temperature showed lower calculated ultimate temperatures for larger column axial force ratios $\bar{p}$. The case in which the ultimate temperature of a column (●) was almost the temperature at which local buckling occurred (◎) (for example, when $\bar{p}$ = 0.1 and L/h = 1) was when local buckling occurred at the upper end of the column when the column suffered overall buckling deformation.

[3] Ultimate temperature values estimated using two-dimensional beam element models (Δ) and

three-dimensional plate element models (●) were more different for columns of larger beam to column ratio L/h and larger width to thickness ratio B/t. The difference was likely attributable mainly to the occurrence of local buckling. Columns with small L/h and B/t were estimated to collapse by overall buckling in both analyses. On the other hand, columns with large L/h and B/t values were estimated to collapse by local buckling when analyzed using three-dimensional plate models. Figure 15(c) shows that the difference in estimated ultimate temperature was as large as about 130°C for the column of B/t = 33.

[4] When the column axial force ratio $\overline{p}$, the width to thickness ratio B/t and the joint translation angle (= L/h) increased, the column was exposed to larger loads and several structural conditions, and both the temperature at which local buckling occurred and ultimate temperature lowered. However, the increases in L/h caused sharp drops in the temperature at which local buckling occurred (◎), and the drop in the numerical ultimate temperature (●) was small. For example, an analysis of a steel column with small B/t and $\overline{p}$, which is widely used in actual frames, is shown in Figures 13(b) and 14(b), revealing that local buckling occurred when the heated span was long (◎) but the ultimate temperature (●) changed little even at large L/h. Columns of small width to thickness ratios (B/t) showed small drops in strength after local buckling, resulting in small drops in ultimate temperature. This trend was also estimated from Equation (1), which gives the residual strength of a steel column after local buckling. Equation (1), which is shown with a thick line Figure 6, shows that compressive strain ε in the section of a column increased largely by an elongating beam but the corresponding drops in residual strength were small at large ε values. Thus, the ultimate temperature of a column is likely to be a stable index of fire resistance, which is little affected by elongation of beams during fire.

[5] The theoretical ultimate temperatures (T_B, T_{LB1} and T_{LB2}) of a column were investigated. The ultimate local buckling temperature T_{LB1} of steel columns in the Recommendation for Fire Resistant Design of Steel Structures (dashed line) and the theoretical buckling temperature T_B (dotted line) were constant regardless of the beam to column ratio (L/h). On the other hand, the ultimate local buckling temperature determined by the modified Suzuki Equation T_{LB2} (solid line) gradually dropped as L/h increased. The three temperatures were similar in columns with small width to thickness ratios B/t, but differed in columns with large B/t values. The reductions were large in the ultimate local buckling temperature in the Recommendation (T_{LB1}) and in the ultimate local buckling temperature determined by the modified Suzuki equation T_{LB2} (Figure 15). In columns of small L/h values, the temperature of the modified Suzuki Equation T_{LB2} (solid line) was higher than the temperature in the Recommendation T_{LB1} (dashed line); and the relationship was the opposite in columns of large L/h values. In the ultimate temperature estimation method, the range in which T_{LB2} (solid line) is lower than T_{LB1} (dashed line) is when $\varepsilon'_1 > \varepsilon_1$ (= 0.04) (Equation (3)). From Equation (2), this means that a joint transition angle δ_H/h of at least 0.03 is generated in the column. From Figures 13(a) to 15(a), most columns of $\overline{p}$ = 0.1 were within this range, but their ultimate temperatures were high because their original axial forces were small.

[6] When the numerical ultimate temperatures (●) of the columns were compared with their theoretical ultimate temperatures T_B (dotted line), the former was larger than the latter in columns of small L/h values. As described in Section 3.1, the coefficient of effective buckling length γ was assumed to be 1.0 for estimating the theoretical temperature T_B by considering the occurrence of local buckling. However, columns with small L/h values showed no local buckling, and thus their coefficients of effective buckling length should have been smaller than 1.0. The difference caused the calculated temperature to be larger than the theoretical temperature. On the other hand, columns of large L/h values suffered local buckling at their upper and lower ends, loosening the bending constraints at the ends. Columns of even larger

L/h values showed larger drops in strength by local buckling, resulting in calculated ultimate temperatures lower than their theoretical ultimate temperatures T_B. Figures 13 to 15 show that most calculated ultimate temperature values were higher than the ultimate local buckling temperatures T_{LB1} and T_{LB2} (solid and dashed lines). The Recommendation for Fire Resistant Design of Steel Structures assesses the fire resistance of columns after local buckling by making some assumptions, and estimates the ultimate temperatures on the safe side for a number of actually used steel columns.

5. SUMMARY

When beams are heated during fire, the beams elongate and push adjacent columns. The longer the span of the heated beam, the larger the elongation of the beam, and the more vulnerable the adjacent hot columns to local buckling. However, local buckling does not cause the steel column to collapse immediately, but the column keeps stable residual strength after the occurrence of local buckling, making the column advantageous in terms of fire resistance. The temperature at which local buckling occurs is strongly affected by the joint translation angle of the column (= elongation of adjacent beams), and assessing the fire resistance of the column at the temperature may result in underestimation of the load bearing capacity of the column. On the other hand, the ultimate temperatures of columns, which were the upper member temperature at which a column could keep equilibrium of forces, did not much depend on the size of joint translation angles under most structural and loading conditions analyzed and were much higher than the temperatures at which the columns suffered local buckling. This trend was especially clear in columns that consisted of thick plates and received small compressive forces along the action axis, which are widely used at seismic designs of Japanese steel buildings, and the columns showed no large drops in ultimate temperature even with large joint translation angles. Methods proposed for estimating the ultimate temperatures of columns were verified by conducting parametric numerical analyses, and were verified to be reliable.

This study investigated square hollow steel columns of the FA class in structural classification by Japanese seismic codes. Columns of larger width to thickness ratios, such as those of FB class, may show larger drops in strength after local buckling and thus larger drops in ultimate temperature than in columns of FA class, and thus the safety should be thoroughly examined. Excessive elongation of beams during fire may also threaten fire compartment room. Therefore, comprehensive studies on these topics, investigation of local buckling behaviors during fire, and high-temperature experiments of members need to be conducted.

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