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Strength of Steel Frame Structures in Fire

Résistance au feu des structures en acier

Feuerwiderstand von Stahlkonstruktionen

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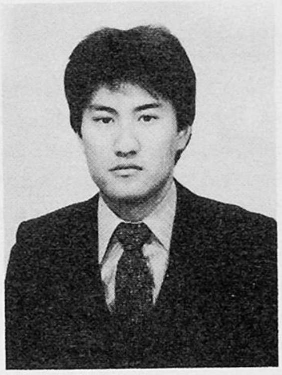

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SUMMARY

The strength of steel frame structures during and after a fire are evaluated using a finite element thermo-visco-elastoplastic analysis based on the original experimental data. Strengthening of the post-fire structure is also described.

RESUME

La résistance des structures en acier pendant et après un incendie est évaluée sur la base de l'analyse thermo-visco-élastoplastique des éléments finis. Le renforcement de la structure après un incendie est aussi décrit.

ZUSAMMENFASSUNG

Der Feuerwiderstand von Stahlkonstruktionen wird mit Hilfe einer auf Finiter Elemente beruhenden Analyse berechnet. Die Verstärkung von Konstruktionen nach einer Feuereinwirkung wird ebenfalls beschrieben.

1. INTRODUCTION

In the case that a steel frame structure is exposed by fire, a strength of the structure decreases during the time fire is spreading. After the fire has been extinguished, a strength of structure called "post-fire strength" is less than the initial design strength before fire, which is called "pre-fire strength".

First, decrease of the strength during fire is calculated by a numerical analysis based on the original test data obtained by the authors. Decrease of the strength is evaluated by a decrease of "degree of safety for the structural collapse".

Second, the relationship between "fire spreading time" and post-fire strength is provided, where the fire spreading time implies total time (counted by minutes) from an outbreak of the fire until a start of fire extinguishing operations.

Third, strengthening of the post-fire structure is performed so as to recover the initial (pre-fire) strength.

2. NUMERICAL CALCULATION

2.1 Thermo-Visco-Elastoplastic Behavior

At an early stage of fire, when the temperature is less than 300 degrees C, deformation of a steel frame structure is caused by a combination of elastic, plastic and thermal strains. When the temperature reaches the 400s, creep strain holds a part of total deformation, and in the 500s - 600s, half of the total deformation is caused by the creep strain. Therefore effect of the creep needs to be included in the numerical analysis of structures in fire.

Steel material in high temperature is assumed here to behave as a thermo-visco-elastoplastic body. It implies that the behavior of steel is described as an independent superposition of a thermo-elastoplastic and a creep behaviors [1].

2.2 Constitutive Relations

For a thermo-visco-elastoplastic body it is assumed that the strain increment de_{ij} can be represented in the form of the sum of elastic, plastic, thermal and creep strain increments, i.e., de^E_{ij}, de^P_{ij}, de^T_{ij}, de^C_{ij}, as

$$de_{ij} = de^E_{ij} + de^P_{ij} + de^T_{ij} + de^C_{ij} \qquad [1]$$

in which each strain increment is described as

$$\begin{aligned} de^E_{ij} &= [(1+p)/E]\, ds_{ij} - (p/E)\, ds_{kk}\,\delta_{ij}, \quad E = E(T, \dot{\bar{e}}) \\ de^P_{ij} &= \Lambda_p\, s^*_{ij} \\ de^T_{ij} &= a\, dT\, \delta_{ij} \\ de^C_{ij} &= \dot{e}^C_{ij}\, dt \end{aligned} \qquad [2]$$

where E is elastic modulus (a function of temperature T and equivalent strain rate $\dot{\bar{e}}$); p, Poisson's ratio; δ_{ij}, Dirac's delta; a, coefficient of thermal expansion; T, temperature; and s^*_{ij}, stress deviator.

Plastic strain increment de^P_{ij} is described as a product of positive scalar function Λ_p and normal to yield surface $\partial f/\partial s_{ij}$, by assuming an associated flow rule; where f is a yield surface. Now the material is a structural steel, and von Mises yield criterion can be applied to the yield surface f, that is, f is describrd as follows by assuming an isotropic strain-hardening;

$$f = \left(\frac{3}{2} s^*_{ij}\, s^*_{ij}\right)^{1/2} - \bar{s}_y(T, \bar{e}^P, \dot{\bar{e}}) \qquad [3]$$

$\bar{e}^P$ but also by temperature T and equivalent strain rate $\dot{\bar{e}}$. Using Prager's consistency condition df=0 to Eq.[3], we get

$$df = \frac{3}{2\bar{s}_y} s_{ij}^* ds_{ij}^* - \frac{\partial \bar{s}_y}{\partial T} dT - \frac{\partial \bar{s}_y}{\partial \bar{e}^P} d\bar{e}^P - \frac{\partial \bar{s}_y}{\partial \dot{\bar{e}}} d\dot{\bar{e}} = 0$$

and by introducing the relations such as

$$s_{ij}^* ds_{ij}^* = \frac{2}{3} \bar{s}_y d\bar{s}_y , \quad d\bar{e}^P = \frac{2}{3} \Lambda_p \bar{s}_y$$

the following relation is finally obtained;

$$d\bar{s}_y - \frac{\partial \bar{s}_y}{\partial T} dT - \frac{\partial \bar{s}_y}{\partial \bar{e}^P} \frac{2}{3} \Lambda_p \bar{s}_y - \frac{\partial \bar{s}_y}{\partial \dot{\bar{e}}} d\dot{\bar{e}} = 0$$

Undefined positive scalar Λ_p in Eq.[2] is thus determined as follows [2].

$$\Lambda_p = \left(d\bar{s}_y - \frac{\partial \bar{s}_y}{\partial T} dT - \frac{\partial \bar{s}_y}{\partial \dot{\bar{e}}} d\dot{\bar{e}} \right) / \left(\frac{2}{3} \bar{s}_y \frac{\partial \bar{s}_y}{\partial \bar{e}^P} \right) \qquad [4]$$

Creep strain increment de_{ij}^C is represented approximately as a product of creep strain rate $\dot{e}_{ij}^C$ and time increment dt as shown in Eq.[2] (see Ref. [3]). Creep strain rate $\dot{e}_{ij}^C$ is described in the following convienient style, by using an equivalent creep strain rate $\dot{\bar{e}}^C$ [4];

$$\dot{e}_{ij}^C = \Lambda_c s_{ij}^* , \quad \Lambda_c = \frac{3}{2\bar{s}} \dot{\bar{e}}^C (T, \bar{s}) \qquad [5]$$

in which $\dot{\bar{e}}^C$ is a function of temperature T and stress level represented by an equivalent stress $\bar{s}$.

2.3 Material Properties

As described in 2.2 , elastic modulus E, equivalent yield stress $\bar{s}_y$ and equivalent creep strain rate $\dot{\bar{e}}$ are treated here as functions of temperature T, equivalent plastic strain $\bar{e}^P$, equivalent strain rate $\dot{\bar{e}}$ and equivalent stress $\bar{s}$. These functions are determined by a series of strain-controlled high-temperature tension tests and stress-controlled high-temperature creep tests organized by the authors [5]. The test specimens are cut from a web of rolled H-beam made by SS41 steel whose nominal tensile strength is 41 kg/mm^2 (4.02 MPa).

2.4 Heat Transfer Analysis [7]

Temperature distribution in the section of non-protected steel frame members are calculated by two-dimensional finite element analysis. Standard temperature-time curve specified for the fire resistance test [6] is used in the calculations, and the heat transfer from heat source to the member is given by a radiation and a convection.

2.5 Structural Analysis [7]

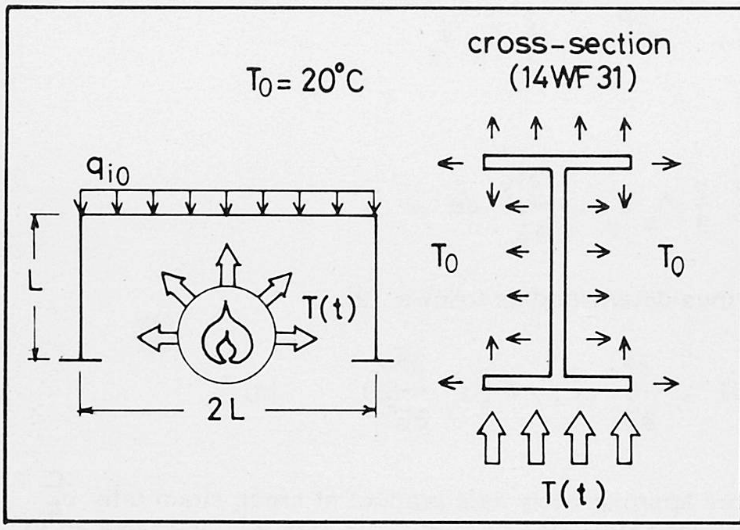
Three-nodes seven-degrees-of-freedom beam-element is used on the basis of the simplest assumptions such as

- small deformation
- in-plane deformation
- Bernoulli-Euler's assumption.

Original Newton-Raphson technique is used for a convergence of iterative calculations.

3. NUMERICAL RESULTS

One-story one-span steel frame structure with fixed ends as shown in Fig. 1 is exposed by fire. The cross-section of the frame member is represented in Fig. 1, where the boundary conditions on radiation and convection are also indicated. Uniform load q_{i0} is applied to a horizontal member of the frame; the magnitude is determined to be 60% of an elastic limit load (first yield load) q_{y0} , for which the yield region occurs at someplace of the members at the first time. The definitions on these loads are also denoted in Fig. 1.



uniform load	definition	q/q_{y0}
q_{i0}	design load or initial load	0.600
q_{y0}	elastic limit load or first yield load	1.000
q_{c0}	collapse load	1.585
q_{c0}/q_{i0}	degree of safety for the structual collapse	2.64

Fig. 1 Dimensions of Steel Frame Structure and Definitions of Initial Uniform Loads q_0.

Thermo-visco-elastoplastic analysis is applied to the structure, and the following calculations are conducted;

(1) Progress of deformation and reduction of strength during fire:

a) Vertical displacement d_v at the center of horizontal member is shown in Fig.2(a) and horizontal displacement d_h at the corner is shown in Fig.2(b), where the fire spreading time t_f is taken as the abscissa.

b) Load-displacement (d_v) curves are shown in Fig.3(a). Using the results, degrees of safety for the structural collapse are indicated in Table 1(a) in the case that fire spreading times t_f are 10, 20, 25 and 30 minutes, where they are defined as the ratios of collapse loads q at t_f=10, 20, 25 and 30 minutes to the initial design load q_{i0}.

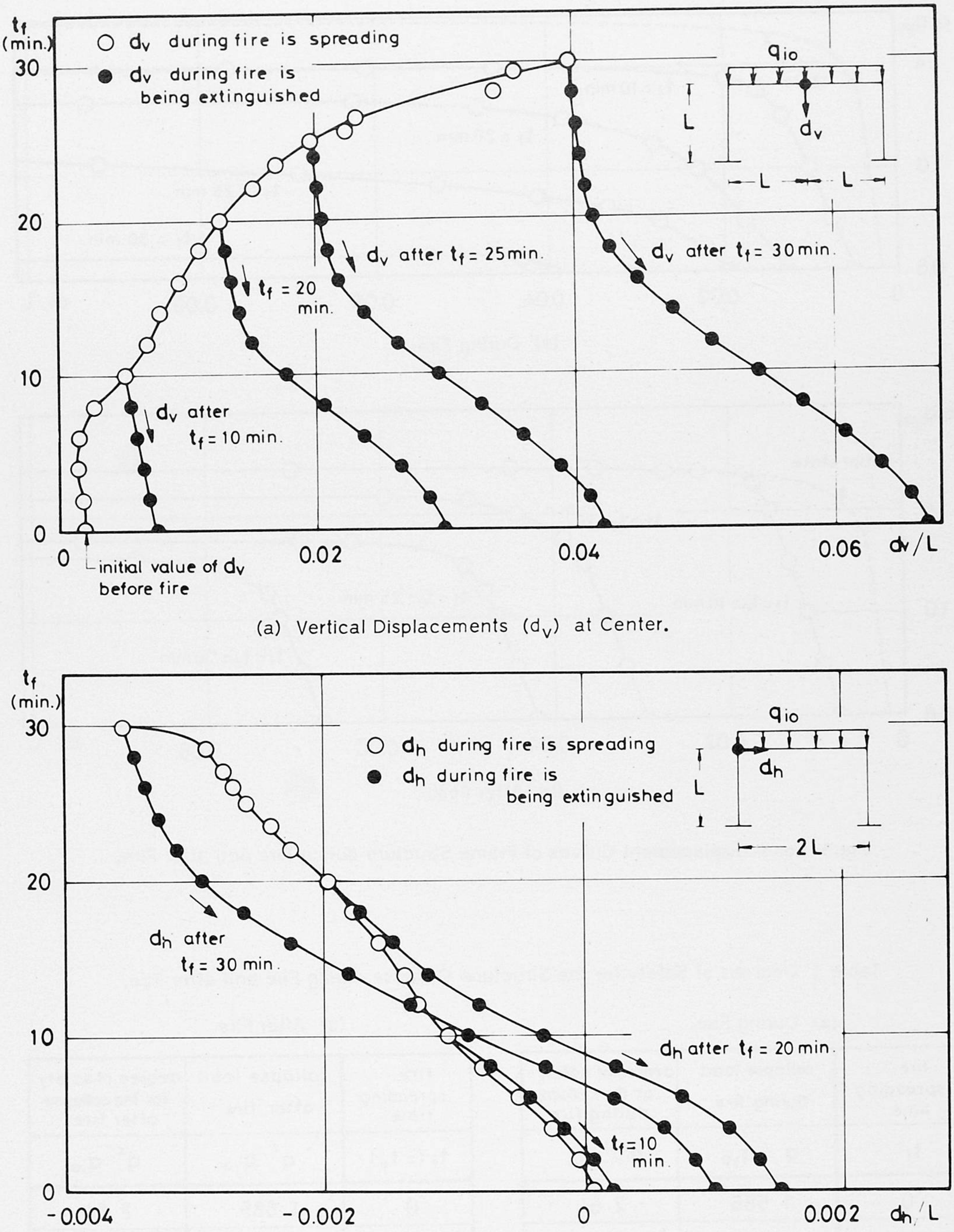
(2) Further progress of deformation and recovery of strength after fire:

a) Displacements d_v and d_h during the time fire is being extinguished are shown in Fig.2(a) and (b), respectively, where the ordinate (t_e) has an opposite direction to the original ordinate (t_f). The time neccessary for fire extinguishing [t_e] is assumed to be equal to the fire spreading time t_f, i.e., $t_e = t_f$.

b) Load-displacement (d_v) curves are shown in Fig.3(b). Using the results, degrees of safety in the case of $t_f = t_e$=10, 20, 25 and 30 minutes are indicated in Table 1(b), where the degrees of safety are defined as the ratios of collapse loads $q*$ at t_e=10, 20, 25 and 30 minutes to the initial design load q_{i0}.

(3) Strengthening of post-fire structure:

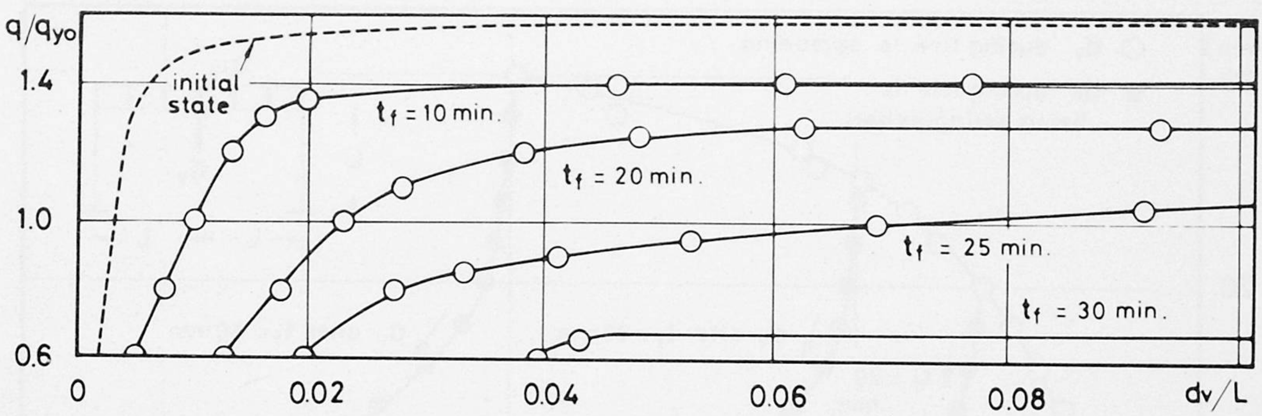
a) Lengthes of strengthening plates, necessary for a recovery of the initial strength after fire, are indicated in Table 2.



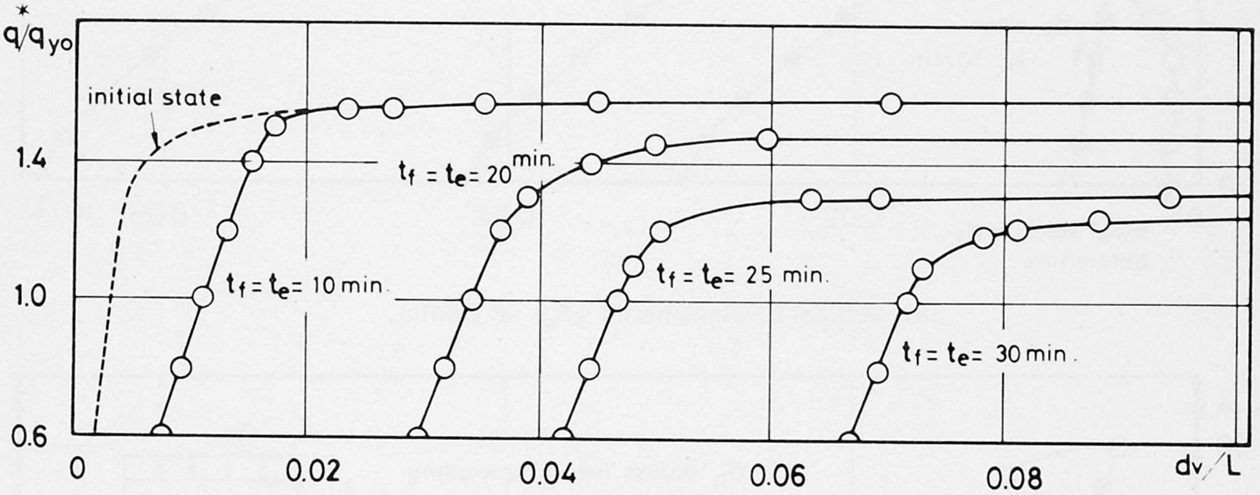
(a) Vertical Displacements (d_V) at Center.

(b) Horizontal Displacements (d_h) at Corner.

Fig.2 Time–Displacement Curves of Frame Structure during Time that Fire is Spreading and that Fire is Being Extinguished.



(a) During Fire.



(b) After Fire.

Fig.3 Load-Displacement Curves of Frame Structure during Fire and after Fire.

Table 1 Degrees of Safety for the Structural Collapse during Fire and after Fire.

(a) During Fire.

fire spreading time	collapse load during fire	degree of safety for the collapse during fire
t_f	q / q_{yo}	q / q_{io}
0	1.585	2.64
10	1.420	2.37
20	1.290	2.15
25	1.050	1.75
30	0.685	1.14

(b) After Fire.

fire spreading time	collapse load after fire	degree of safety for the collapse after fire
$t_f\ (= t_e)$	q^* / q_{yo}	q^* / q_{io}
0	1.585	2.64
10	1.585	2.64
20	1.470	2.45
25	1.320	2.20
30	1.250	2.08

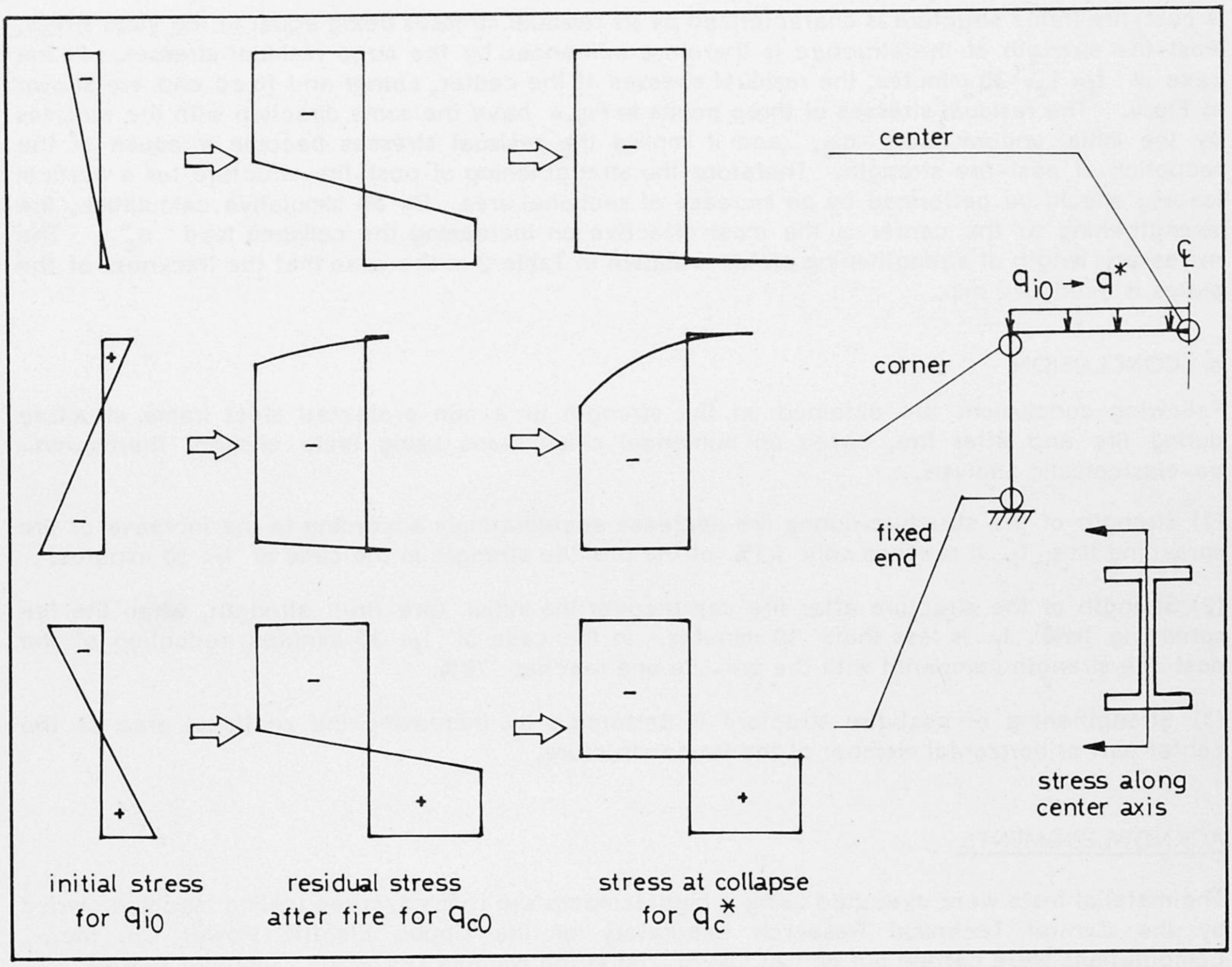
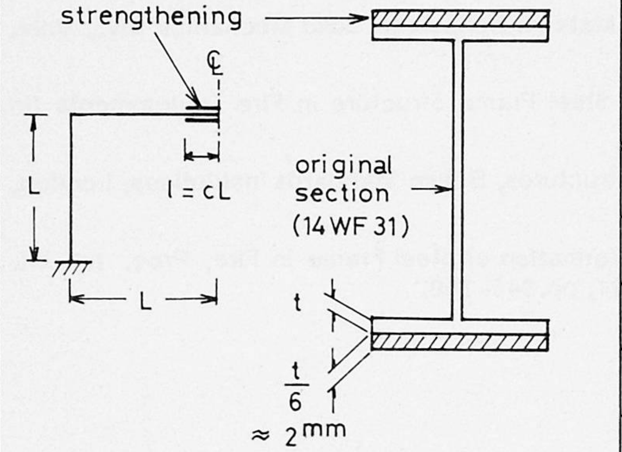


Fig.4 Residual Stresses in H-Section of Frame Structure after Fire ($T_f = T_e =$ 30 minutes).

Table 2 Lengths of Strengthening Plates and Recovery of Strength for the Structural Collapse due to the Strengthening.



fire spreading time	length of strengthening plates	collapse load after strengthening
$t_f(=t_e)$	$c\ (l = cL)$	q^*/q_{y0}
0	0	1.585
10	0	1.585
20	3/32	1.600
25	5/32	1.605
30	9/32	1.595

A post-fire frame structure is characterized by its residual stresses being equal to the yield stress. Post-fire strength of the structure is therefore influenced by the large residual stresses. In the case of $t_f = t_e = 30$ minutes, the residual stresses at the center, corner and fixed end are shown in Fig.4. The residual stresses of three points in Fig.4 have the same direction with the stresses by the initial uniform load q_{i0}, and it implies the resisual stresses become a cause of the reduction of post-fire strength. Therefore the strengthening of post-fire structure for a vertical loading should be performed by an increase of sectional area. By an simulative calculation, the strengthening at the center is the most effective on increasing the collapse load q_c^*. The necessary length of strengthening plates is shown in Table 2 in the case that the thickness of the plates is fixed as 2 mm.

4. CONCLUSION

Following conclusions are obtained on the strength of a non-protected steel frame structure during fire and after fire, based on numerical calculations using finite element thermo-visco-elastoplastic analysis.

(1) Strength of the structure during fire decrease acceleratingly according to the increase of fire spreading time t_f. It reaches only 43% of the pre-fire strength in the case of $t_f = 30$ minutes.

(2) Strength of the structure after fire can recover the initial (pre-fire) strength, when the fire spreading time t_f is less than 10 minutes. In the case of $t_f = 30$ minutes, reduction of the post-fire strength compared with the pre-fire one reaches 79%.

(3) Strengthening of post-fire structure is performed by increasing the sectional area of the center part of horizontal member of the frame structure.

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